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SEISMIC DESIGN CRITERIA FOR SOIL LIQUEFACTION

by

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The Navy has numerous bases located in seismically active regions throughout the world. Safe and effective structural design of waterfront facilities requires calculating the expected site specific ground motion and determining the response of these complex structures to the induced loading. The Navy's problem is further complicated by the presence of soft saturated marginal soils which can significantly amplify the levels of seismic shaking and liquefy as evidenced in the 1989 Loma Prieta earthquake. Liquefaction is a major factor at the waterfront and most of the damage the Navy has sustained from earthquakes can be attributed to it. The presence of unconsolidated loose cohesionless soils and the high water table makes waterfront sites especially vulnerable. This report establishes liquifaction criteria suited for the deisgn of new facilities and upgrade of existing facilities. The criteria developed herein presents reasonable performance standards balancing performance and damage minimization against the cost of implementation.

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Executive Summary

The Navy has numerous bases located in seismically active regions throughout the world. Safe and effective structural design of waterfront facilities requires calculating the expected site specific ground motion and determining the response of these complex structures to the induced loading. The Navy's problem is further complicated by the presence of soft saturated marginal soils which can significantly amplify the levels of seismic shaking and liquefy as evidenced in the 1989 Loma Prieta earthquake. Liquefaction is a major factor at the waterfront and most of the damage the Navy has sustained from earthquakes can be attributed to it. The presence of unconsolidated loose cohesionless soils and the high water table makes waterfront sites especially vulnerable. The Loma Prieta Earthquake of October 17, 1989 caused $125 million dollars of damage to U. S. Navy facilities. The predominant cause of damage was liquefaction of cohesionless waterfront deposits. The Navy again sustained extensive liquefaction during 1993 Guam earthquake in the amount of approximately $150 million.

Liquefaction is a process in which the seismic shear waves cause an increase in the pore water pressure in a cohesionless soil strata. This increase in pore pressure reduces the effective stress confining the soil. The reduction in effective confining stress causes a reduction of shear modulus of the soil, which in turn, results in increased soil deformation. Also associated with liquefaction is a loss in bearing strength. In the case of full liquefaction, when the increase in pore water pressure reduces the confining stress to zero, the soil experiences a full loss of strength and undergoes large viscous deformations. Large lateral deformations are possible when liquefaction occurs on ground having even minimal slope. This is a major continuing problem faced by the Navy because mission required facilities must be situated at the waterfront often on marginal soils.

Safe and effective seismic design requires establishment of performance goals, specification of the earthquake load levels and given that loading, definition of the expected acceptable structural response limits. Both Navy and civilian codes have made distinctions between ordinary and essential construction. Generally, essential construction is expected to be operational after an earthquake. Facilities are deemed as essential by virtue of their need after an earthquake such as a hospital, fire station, or emergency recovery center. Navy facilities may be deemed essential by their mission requirement in support of national defense, such as a communication station. Piers and wharves are deemed as essential based on the needs for fleet operability. The decision to declare a structure as essential is to be made by the user in conjunction with the Naval Facility Engineering Command design agent. This report establishes liquefaction criteria suited for the design of new facilities and upgrade of existing facilities. The criteria is summarized in the following table. Because the potential for liquefaction is so extensive at the waterfront, economics must be considered for both new and remedial projects. While it may be very difficult to prevent liquefaction in all cases for high levels of ground shaking, the extent of vertical and horizontal deformation and the resulting disruption to port operations must be considered. Widespread disruption is very possible as evidenced by the 1995 Kobe earthquake. The criteria developed herein presents reasonable performance standards balancing performance and damage minimization against the cost of implementation.

Liquefaction Criteria Summary For Seismic Design

	Ordinary Buildings	Piers & Wharves	Essential Buildings	Hazardous or Polluting Construction
Navy Provisions	NAVFAC P355		NAVFAC P355.1	
Performance Objectives	Minor earthquake no damage Moderate earthquake no structural damage Major earthquake no collapse	Level 1 no structural damage Level 2 controlled inelastic behavior, repairable	Level 1 minor damage no loss of function Level 2 repairable damage	Prevent release of materials breach, or loss of contents
Design Earthquake (Exceedance Probability) **Structure**	50% 50 years	Level 1 50% 50 years Level 2 10% 50 years	Level 1 50% 50 years Level 2 10% 100 years	10% 250 years
Liquefaction	Level 1 50% 50 years Level 2 10% 50 years	Level 1 50% 50 years Level 2 10% 50 years	Level 1 50% 50 years Level 2 10% 100 years	10% 250 years
Design Response Limits **Structural**	Code stress and drift limits	ductility limits	ductility & drift limits	no collapse/breach limits
Liquefaction	Level 1 FS 1.5 Level 2 FS > 1.0 controlled deformation	Level 1 FS 1.5 Level 2 FS > 1.0 controlled deformation	Level 1 FS 1.5 Level 2 FS > 1.1 controlled deformation	FS > 1.1 controlled deformation

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Introduction

Liquefaction is a major Navy problem at the waterfront and is probably the single greatest damage causing mechanism. Safe effective seismic design consists of establishment of performance goals, specification of the earthquake loading and given that loading, definition of the expected acceptable structural response limits.

Both Navy and civilian codes have made distinctions between ordinary and essential construction. Generally, essential construction is expected to be operational after an earthquake. Facilities are deemed as essential by virtue of their need after an earthquake such as a hospital, fire station, or emergency recovery center. Navy facilities may be deemed essential by their mission requirement in support of national defense, such as a communication station. Piers and wharves are deemed as essential based on the needs for fleet operability.

The decision to declare a structure as essential is to be made by the user in conjunction with the Naval Facility Engineering Command design agent.

When considering the potential for liquefaction associated with an essential function it is critical that the consequences of the liquefaction occurrence and its effect on the function be considered. The function may be a structure or a lifeline.

Performance Objectives

The following performance objectives are presented herein and are proposed for Navy use. They are based on mandates of public law and extensions of current Navy criteria.

Ordinary Construction - Liquefaction associated with construction categorized as "ordinary" shall be evaluated to insure the level of performance is maintained. In general ordinary construction is expected to:

- Resist a minor level of ground motion without damage;
- Resist a moderate level of ground motion without structural damage, but possibly experience some nonstructural damage;
- Resist a major level of earthquake (10 percent probability of exceedance in 50 years) ground motion without collapse, but with structural as well as nonstructural damage.

Piers and Wharves- Liquefaction assessments associated with piers and wharves shall be evaluated to insure the level of performance is maintained. In general pier and wharf construction is expected to:

- To resist earthquakes of moderate size, Level 1, which can be expected to occur one or more times during the life of the structure without structural damage of significance.

- To resist major earthquakes, Level 2, which are considered as infrequent rare events maintaining life safety, precluding total collapse but allowing a measure of limited controlled inelastic behavior which will require repair.

Essential Construction - Liquefaction evaluation associated with construction categorized as "essential" shall be evaluated to insure the level of performance is maintained. In general essential construction is expected to:

- Resist the maximum probable earthquake likely to occur one or more times during the life of the structure (50 percent probability of exceedance in 50 years) with minor damage without loss of function and the structural system to remain essentially linear.
- Resist the maximum theoretical earthquake with a low probability of being exceeded during the life of the structure (10 percent probability of exceedance in 100 years) without catastrophic failure and a repairable level of damage.

Hazardous Materials - Liquefaction associated with construction categorized as associated with "hazardous materials" shall be evaluated to insure the level of performance is maintained. In general construction related to containment of hazardous materials is expected to:

- Conform with criteria for essential construction
- Resist pollution and release of hazardous materials for an extreme event (10 percent probability of exceedance in 250 years)

Seismic Loads For Liquefaction Evaluation

The following is based on current Navy criteria and an extension of existing mandates logically applied to analogous situations. Navy construction shall be designed to resist the loading produced as follows:

- **Ordinary category of construction on average seismicity sites**
  For sites of average seismicity, use NAVFAC P355 provisions, which establish the earthquake at a nominal 10 percent probability of exceedance in 50 years. For liquefaction evaluation use a Level 1 earthquake having a 50 percent probability of exceedance in 50 years and a Level II earthquake having a 10 percent probability of exceedance in 50 years.

- **Wharves and Piers**
  Design of wharves, wharf dikes, and piers shall use a two-earthquake procedure with a Level I earthquake having a 50 percent probability of exceedance in 50 years and a Level II earthquake having a 10 percent probability of exceedance in 50 years based on a local site seismicity study. Values less than NAVFAC P355 are not be permitted.

- **High seismicity or essential category of construction**
  Sites of high seismicity controlled by local faulting where general NAVFAC P355 provisions do not account for the local hazard potential, or where the structure is deemed important and essential shall use a two-earthquake procedure with a Level I earthquake having a 50 percent probability of exceedance in 50 years and a Level II earthquake having a 10 percent probability of exceedance in 100 years based on a local site seismicity study. Values less than NAVFAC P355 are not be permitted.

- **Construction containing polluting or hazardous material**
  An earthquake with a 10 percent probability of exceedance in 250 years exposure shall be used.

As part of this criteria:

- the determination of the design earthquake shall as a minimum be performed using techniques described in NFESC TR-2016-SHR or other equivalent procedures.

In addition to seismic ground motion there are additional hazards which must be considered:

- Fault movement and local ground displacement
- Liquefaction and associated lateral spreading, settlement flow slides, loss of support and buoyancy of buried tanks.
- Landslides
- Tsunamis

Liquefaction

Design of structures shall include provisions to evaluate and resist liquefaction of the foundation and account for expected potential settlements and lateral spread deformation.

As part of this criteria:

- Retaining structures shall as a minimum be designed using provisions in NCEL Technical Report R-939.

- Liquefaction and lateral spread shall as a minimum be computed based on guidance in NCEL Technical Note 1862.

Special care will be given to buried pipelines in areas subject to liquefaction to preclude breaks resulting in release of hazardous materials. The most important element in seismic

design of pipelines is proper siting. It is imperative to avoid areas of landslide and lateral spread.

The presence of any potentially liquefiable materials in foundation or backfill areas shall be fully analyzed and expected settlements computed. Specific attention shall be paid to the acceptability of the amount of settlements. Since liquefaction is a major damage mechanism at the waterfront, remediation is a mandatory requirement where the risk of a release of hazardous materials as shown by computation is possible such as in a pipeline break or tank failure. Liquefaction of backfil around dry-docks is a potentially critical problem which must be evaluated.

Ground Deformation and Factor of Safety

The presence of any potentially liquefiable materials in backfill areas shall be fully analyzed and expected settlements computed. Specific attention shall be paid to the acceptability of the amount of settlements.

Ordinary Construction- Liquefaction is to be precluded under minor earthquakes but may occur under a major earthquake as long as collapse is precluded.

- The Factor of Safety against liquefaction for the Level 1 earthquake shall be greater than 1.5.

- The Factor of Safety against liquefaction for the Level 2 earthquake should be greater than 1.0 with settlements of about 4 inches or less and lateral deformations of about 6 to 12 inches or less. Where it may not be possible to achieve a Factor of Safety greater than 1.0 for the Level 2 earthquake, a Factor of Safety greater than 0.9 may be considered as long as the computed deformation state is shown to have limited controlled settlements and lateral spread equivalent to the values stated so as to prevent collapse of the structure.

Wharves and Piers- Under Level 1 earthquakes large deformations resulting in widespread pavement disruption in adjacent areas should be avoided where economically feasible.

- For a Level 1 earthquake, the Factor of Safety against liquefaction in the backfill should be 1.5 or higher with settlements of about 1 inch or less and lateral deformations of about 3 inches or less.

- For a Level 2 earthquake, the Factor of Safety against liquefaction in the backfill should be 1.0 or higher with settlements of about 4 inches or less and lateral deformations of about 6 to 12 inches or less. Where it may not be possible to achieve a Factor of Safety greater than 1.0, a Factor of Safety greater than 0.9 may be considered as long as the computed deformation state is shown to have limited controlled settlements and lateral spread equivalent to the values stated.

High seismicity or essential category of construction- Under Level 1 earthquakes large deformations resulting in widespread pavement disruption should be avoided where economically feasible.

- For a Level 1 earthquake, the Factor of Safety against liquefaction in the foundation or backfill should be 1.5 or higher with settlements of about 1 inch or less and lateral deformations of about 3 inches or less.

- For a Level 2 earthquake, the Factor of Safety against liquefaction shall be 1.1 or higher with settlements restricted to preclude major nonrepairable structural damage. Computed deformation state shall be shown to have limited controlled settlements and restricted lateral spread..

Construction containing polluting or hazardous material- The Factor of Safety against liquefaction shall be 1.1 or higher with settlements restricted to preclude a breach of the containment structures. Computed deformation state shall be shown to have limited controlled settlements and restricted lateral spread..

Response Of Dikes and Retaining Structures

The general response of a waterfront structure under the design earthquake levels shall be:

- For a Level 1 earthquake, essentially elastic response is required throughout the structure.

- For a Level 2 earthquake, limited controlled inelastic behavior

Wharf Dike- For a wharf, design of the under-wharf dike retaining structures as a minimum shall have permanent horizontal deformation of the slope computed by a Newmark analysis and such deformation shall not exceed:

- For a Level 1 earthquake, 4 inches

- For a Level 2 earthquake, 12 inches

Anchored Sheetpile Retaining Walls- Design of anchored sheetpile retaining walls shall limit permanent displacement at the top of the sheetpile to the following

- For a Level 1 earthquake, less than 1 inch

- For a Level 2 earthquake, less than 4 inches

Design of sheet pile bulkheads, dikes and retaining structures shall include provisions to resist full liquefaction of the backfill and for expected potential lateral spread deformation. Retaining structures shall as a minimum be designed using provisions in NCEL Technical Report R-939. Liquefaction and lateral spread shall as a minimum be computed based on guidance in NCEL Technical Note 1862.

All crane rails shall be supported on piles including the seaward and the landward rail. The crane rails shall be connected horizontally by a continuous deck, beam or other means to control the gage of the rails and prevent spreading. The rails shall be grounded. For corrosion protection, it is advantageous to insulate the reinforcing steel in the piles from that in the deck.

Existing Construction

Seismic reviews of existing waterfront construction directed by requirements of the Naval Facilities Engineering Command shall utilize the above criteria for new construction as the target requirement for upgrade. The requirement for evaluation of the seismic resistance and possible upgrade is triggered when the loading on the structure changes such as when the mission of the structure is changed or when the structure requires major repairs or modifications to meet operational needs. When it is shown to be impossible or uneconomical to achieve new construction levels of performance, an economic life cycle cost analysis shall be performed to determine the most cost effective level of seismic design upgrade. Various alternative upgrade levels shall be considered ranging from the existing condition to the maximum achievable. Each alternative shall be examined to determine the cost of the upgrade, the cost of expected earthquake damage over the life of the structure and the impact of the damage on life safety, operational requirements, and damage to the environment. The choice of upgrade level shall be made by the design team based on a cost effective strategy consistent with requirements of life safety, operational needs and protection of the environment.

Supporting Technical Information

Seismic Design Criteria

For

Soil Liquefaction

Supporting Technical Information
on
Seismic Soil Liquefaction Criteria

Introduction

The Navy has numerous bases located in seismically active regions throughout the world. Safe, effective design of waterfront structures requires calculating the expected site specific earthquake ground motion and effective design of complex waterfront structures. The Navy's problem is further complicated by the presence of soft saturated marginal soils that can significantly amplify the levels of seismic shaking and liquefy as evidenced in the 1989 Loma Prieta earthquake. The Navy began its seismic program in response to the 1977 Earthquake Hazards Reduction Act. Executive Order 12699 reinforces the commitment to earthquake safety.

The Loma Prieta Earthquake of October 17, 1989 caused $125 million dollars of damage to U. S. Navy facilities. The predominant cause of damage was liquefaction of cohesionless waterfront deposits. Liquefaction is a process in which the seismic shear waves cause an increase in the pore water pressure in a cohesionless soil strata. This increase in pore pressure reduces the effective stress confining the soil. The reduction in effective stress causes a reduction in the shear modulus of the soil, which in turn, results in increased soil deformation. Also associated with liquefaction is a loss in bearing strength. In the case of full liquefaction, when the increase in pore water pressure reduces the confining stress to zero, the soil experiences a full loss of strength and undergoes large viscous deformations. Large lateral deformations are possible when liquefaction occurs on ground having even minimal slope. This is a major continuing problem faced by the Navy because mission required facilities must be situated at the waterfront often on marginal soils.

Observation of the US Naval Station, Treasure Island acceleration record from the 1989 Loma Prieta Earthquake shows that at about 15 seconds after the start of recording, the ground motion was subdued; this was probably caused by the occurrence of subsurface liquefaction. Liquefaction occurred after about 4 or 5 "cycles" of shaking, about 5 seconds of strong motion. Sand boils were observed at numerous location and bayward lateral spreading occurred with associated settlements. Ground cracking was visible with individual cracks as wide as 6 inches. Overall lateral spreading of 1 foot was estimated. Ground survey measurements indicate that settlements of 2 to 6 inches occurred variably across the island and that some areas had as much as 10 to 12 inches of settlement. The liquefaction related deformations resulted in damage to several structures and numerous

broken underground utility lines, Egan (1991). The Navy again sustained extensive liquefaction during 1993 Guam earthquake.

When a loose sand is subjected to seismically induced vibratory motion, it tends to decrease in volume. If it is saturated and drainage is impeded, some of the interparticle stress is transferred to the water. The transferred load causes a rise in the pore water pressure; generally, the higher the intensity of vibration, the greater the potential for increase in pore water pressure. As the pore pressure approaches the confining pressure on the soil, shear resistance is lost. As a consequence a structure situated on this soil may tilt and settle, resulting in differential motions which may cause severe damage. Increased lateral loads from liquefied soil can result in failures of waterfront retaining structures.

Early quantitative studies of liquefaction pertained to natural earth slopes which became unstable from a gradual rise in the water table or tidal fluctuations which caused excess seepage pressures. Generally, a massive flow slide would begin, and the soil came to rest only when the slope angle had been reduced to a few degrees. To explain this phenomenon Casagrande (1916) proposed the “critical void ratio” concept. Subsequently, following extensive studies of numerous flow slides along the banks of the Mississippi River, empirical rules were developed by the Corps of Engineers to predict the likelihood of occurrence of such flow slides. During the last 25 to 30 years, the term “liquefaction” has been extended to include soil behavior under cyclic loading conditions caused by earthquake vibrations. While the end result - loss of soil strength - is the same whether caused by static or dynamic loading, the shear stresses leading to liquefaction under cyclic loading conditions may be much lower than those required to cause liquefaction under static loading conditions. Under continuous vibrations cyclic stresses cause an incremental buildup of pore pressure which progressively reduces the effective strength. Soil borings normally taken at a site provide information on existing soil conditions. With proper analysis, this soil data can give an indication of the liquefaction potential in earthquake-prone regions.

The strength that a sand can mobilize to resist shearing along a given plane depends on the effective or intergranular pressure on the plane and the effective coefficient of friction. The shearing resistance or strength τ_f may be written

$$\tau_f = \sigma' \tan \phi'$$

in which σ' is the effective stress and ϕ' is the effective angle of internal friction. In a saturated sand the intergranular normal stress σ' is defined as

$$\sigma' = \sigma - u$$

where

σ	Total normal stress
u	Pore water pressure

Then

$$\tau_f = (\sigma - u) \tan \phi'$$

If the porewater pressure, u, increases, while the total stress σ remains constant, the shear strength τ_f across any plane of failure decreases independent of the friction angle ϕ'. When $u = \sigma$, then $\tau_f = 0$, and the sand has lost all its shear strength and is said to have liquefied. The sand is sometimes considered to have liquefied when large strains occur under applied loads. In soil mechanics practice, the term "soil liquefaction" may be defined by two criteria. One defines liquefaction in terms of loss of strength and material transformation of a granular material into a fluid. An alternate definition is expressed in terms of the amount of strain or deformation that is unacceptable from a structural viewpoint.

Site Definition

A detailed site investigation must, as a minimum, provide information on the type and in situ condition of the soil as a function of depth and the location of the water table so that a soil profile may be constructed. The extent of the investigation is controlled by the importance of the structure and the planned investigation methods. For conventional waterfront structures of normal importance where large scale soil test programs are not possible, it is suggested that at least standard penetration tests (SPT) or some other technique for determining in situ strength or density be used in conjunction with the recovery and classification of samples. The SPT blow count data may be used to evaluate liquefaction potential as will be shown below.

A boring log can be constructed by identifying the major soil types, showing blow counts, soil index tests, soil classification, and depth of water table. The nature of the site will dictate the number of tests required. After the general profile of the site has been made and the types of soils present identified, it may be more economical to use more expedient investigative techniques such as the static cone penetrometer (friction cone) to define larger regions. This latter test can be performed much quicker than the standard penetration test; however, samples are not taken. Correlations exist between friction cone resistance and standard penetration resistance.

In order to properly identify the in situ soils, tests should be performed on recovered samples. Visual classification and Atterberg limits can be used to identify cohesive soils that will not liquefy in a traditional sense (although they may undergo large strains). The Unified Classification of soils (see abbreviated summary in Table 1) has been found to be very useful by engineers for classifying soil types.

The extent to which other forms of sampling and laboratory tests are performed depends on the nature of the site and size of the project. Cyclic triaxial tests may not be

meaningful unless performed on undisturbed samples. This is particularly true when the sample exhibits cementation, a definite structure, or is interbedded with thin lenses of different materials. In modest programs of site definition, emphasis must be placed on standard penetration tests rather than laboratory triaxial tests. Triaxial tests on undisturbed samples may be required when soils differ considerably from those that have been already tested and reported in the literature.

Factors Affecting Liquefaction

The major factors associated with the liquefaction of saturated cohesionless soils are: initial relative density, cyclic shear stress level, initial (static) shear stress level, initial effective confining pressure, drainage conditions, and the number of cyclic shear stress applications, or duration of shaking. Of additional importance are fines content and soil grain characteristics such as particle size, shape, and gradation. Soil structure, the fabric as a result of previous history, is known to be a significant parameter, but it is difficult to define or sometimes even recognize and, hence, its effects are difficult to quantify.

The foregoing factors reflect the physical properties of the soil, the initial stress conditions, stratigraphy in the ground, and the characteristics of the applied earthquake motions. Many of these items are difficult to control precisely in the laboratory and impossible to evaluate reliably in the field. A brief discussion follows on some of the more significant factors affecting liquefaction.

<u>Dynamic Shear Stress Level</u>: The fundamental concept of liquefaction is based upon the coupling of shear strain and volumetric strain exhibited by soils. The process of pore pressure buildup, leading to liquefaction under cyclic loading, is dependent upon the volumetric strain response under applied shear stresses. The residual increment of pore water pressure generated by an applied dynamic shear stress cycle is, under undrained conditions, related to the shear strain which is, in turn, related to the magnitude of that stress cycle. Actual earthquake motions may have components in all three principal directions. The most critical stresses from a liquefaction viewpoint arise from vertically propagating horizontal shear waves. Vertical stress components are not considered significant since these are of a dilatational nature and completely absorbed by the pore water.

<u>Dynamics of Earthquake Shear Stress</u> Earthquake ground motions generally consist of a number of randomly distributed peak stress cycles of varying shapes and magnitudes. Difficulties involved in analyzing the various random earthquake ground motions have led to an attempt to express earthquake records in terms of an equivalent number of uniform stress cycles (Lee and Chan, 1972). The number of significant cycles in a particular earthquake record depends directly upon the frequency content and the duration of loading. These, in turn, are related to the magnitude of the earthquake, the distance to its epicenter, and the nature of the materials through which the stress waves must propagate.

It has been noted by Peacock and Seed (1968) and Yoshimi and Oh-Oka (1975) that the frequency of vibration, at least within 0.17 to 12 cps, which covers the range of earthquake motions, at least in overburden, is of secondary importance. The actual shape of the stress pulse used in laboratory test simulations has been found not to be critical; i.e., whether or not it is in the form of a sine wave, a saw tooth, or other form. It is common to present soil susceptibility to liquefaction in terms of number of uniform stress cycles causing liquefaction under a specified level of applied shear stress. The number of stress cycles a specimen can withstand increases almost exponentially with a decrease in shear stress level for any constant confining stress level and relative density.

There are some weaknesses in simulating random earthquake motions in terms of uniform cycles. For example Martin, Finn and Seed (1975) note that the tendency for dry sands to undergo volume changes is a direct function of dynamic shear strain level. But dynamic shear strain level is a function of soil modulus of rigidity G, which in turn depends upon the effective confining stress level and, hence, the pore water pressure generated. Since the pore pressure level existing at the time of application of a specific peak is very important, the relative position of any peak in a sequence of loading cycles is significant. Consideration of the effects of stress reversals also suggests that the peculiar characteristics of the loading history (i.e., the symmetry of the stress record, etc.) may be significant. Ishihara, Tatsuoka and Yasuda (1975) note that ground motion inputs in which the maximum peak occurs early are less critical than input records for which the peaks are more uniformly distributed (i.e., vibratory as opposed to shock loadings).

Relative Density The relative density of a soil is one of the major factors regarding liquefaction potential of cohesionless sands. Relative density is stressed here rather than absolute density since it is actually the pore volume of the soil compared to its minimum and maximum possible pore volumes that is of significance. The denser a soil, the lower is its tendency toward volume contraction during shearing; the lower is the pore pressure which will be generated; hence, the more unlikely to liquefy.

Relative density can be controlled in the laboratory using reconstructed samples; however, in typical field situations with complex stratification, relative density may lose its meaning. A factor such as relative density has meaning only in uniform soil conditions; actual experience shows that natural soil deposits are quite often very heterogeneous.

It is also conceivable that there is an upper limit of relative density, D_R, above which a soil under field behavior will either no longer tend to compress and generate pore pressures or will, immediately upon commencing yielding, undergo volume increases which prohibit liquefaction. Based on specific site data taken from the 1964 Niigata earthquake, Kishida (1969) concludes that these soils are not likely to liquefy at relative densities above 75 percent. Although cyclic mobility (temporary loss of strength) can occur at relative densities up to 100 percent, it is thought that negligible distortions occur in this range at least prior to any drainage or pore water redistribution (Castro and Poulos, 1976). It is impossible to define an upper limit to D_r beyond which liquefaction will not occur; nevertheless, it appears it is less probable for a value of D_r above about 80 percent.

Initial Effective Confining Stress The resistance of a soil to liquefaction under cyclic loading has been noted to be a function of the effective confining pressure, prior to application of shear. Field observations of liquefaction of level ground have generally been limited to relatively shallow depths, in few cases below 50 or 60 feet. This was noted by Kishida (1969) who observed in the 1964 Niigata earthquake that liquefaction did not occur where effective overburden stress exceeds 2 kg/cm^2 (27 psi). Although there is a trend toward reduced liquefaction potential at higher stresses, the observed field cases are very limited and cannot be expected to apply in all situations. Liquefaction evaluations must not omit regions simply because the effective pressure exceeds some empirical value.

Because it is difficult to estimate lateral stress levels in the field, the vertical effective stress is used to define the level of confinement, but much work is available (Seed and Peacock, 1971) to indicate that the ratio of lateral to vertical stress K_0 and, hence, the true degree of confinement actually existing in the field are of major importance.

The shear stress level required to cause liquefaction in remolded sand specimens at a relative density less than 80 percent has been found to vary linearly with confining stress levels (Seed and Lee, 1966, and Peacock and Seed, 1968). Therefore it has been found convenient to normalize the effects of dynamic cyclic shear stress level with the value of initial effective confining stress. It is important to recognize that the use of this normalized ratio may not always be applicable to field conditions, particularly where strongly developed structure or cementation is present. Thus, this simplification in treatment of liquefaction potential may not be valid in all circumstances. Soils near the ground surface, under very small degrees of confinement could have resistance to liquefaction in excess of that suggested from test results acquired at higher confining stress levels. This might be associated with material fabric or structure, or, in effect, equivalent to a previous stress history or over-consolidation pressure. That this exists for hydraulic fill sands has been suggested by Meehan (1976),

Drainage Conditions The rate at which pore water pressure is permitted to dissipate from within a soil body has a major influence upon whether or not liquefaction can occur, particularly under cyclic loading (Wong, Seed, and Chan, 1974). Since the rate of pore pressure dissipation is known to be a function of the square of the longest drainage path, the detailed geometry of the soil profile is important. A study of the interrelationships between different layer compressibilities and permeabilities on the occurrence of liquefaction has been presented by Yoshimi and Kuwabara (1973). This analytical study, based upon solutions to the Terzaghi one-dimensional consolidation problem, illustrates that liquefaction will propagate easily from a lower liquefied layer to an overlying permeability than the initially liquefied striation.

A useful tool for investigating the influence of drainage on potentially liquefiable soil strata is discussed by Seed, Martin and Lysmer (1975). Effective stress computer codes provide a numerical solution of the diffusion equation with a pore pressure-generating term included to represent the earthquake-generated pore-pressure increases.

It is possible to investigate the influence of length of drainage path, stratification, water table and saturation level variations, different permeabilities, compressibilities, densities, and other conditions.

Grain Size Characteristics Limits on gradation curves can define bounds separating liquefiable and non-liquefiable soils. The lower boundary on particle size shows the influence of the fines in decreasing the tendency of the soils to density. Plastic fines make more difficult for the sand particles to come free of each other and seek denser arrangements, (NRC 1985). Fines content has been shown to be a factor in the occurrence of liquefaction and is delineated in field prediction relationships. The upper boundaries are significant because they are associated wit the permeability of coarser material. Thus, increased drainage and dissipation of pore pressure can occur. Both the grain size and distribution can control the pore pressure buildup and dissipation

Previous Stress History The influence of previous stress history is of major interest in liquefaction studies. Finn, Bransby and Pickering (1970) present laboratory data showing that a sample, which has previously liquefied, is more susceptible to liquefaction. A specimen of sand at an initial relative density of 50 percent and an initial effective isotropic confining pressure of 200 kN/m^2 was subjected to cyclic loading with stress reversals. The specimen first underwent limited flow or cyclic mobility under the extensional portion of the 25th load cycle. This specimen then underwent several additional cycles wherein it reliquefied, flowed, and then restablilized. After a total of 29 load cycles, the specimen was permitted to drain, and was reconsolidated under an effective spherical pressure of 200 kN/m^2, which yielded a relative density of 60 percent. Upon resumption of cyclic loading the specimen was noted as reliquefying during the extensional segment of its first loading cycle, in spite of its increased relative density value over that of the initial test sequence. Based on such information, it is possible that the number of loading cycles required to cause liquefaction is substantially reduced by previous episodes of liquefaction. The conclusion is that judgment is necessary in interpreting liquefaction potential of sites which underwent previous liquefaction.

Parameters Indirectly Affecting Liquefaction

There is a family of soil parameters which, while not related to the liquefaction process directly, do influence the liquefaction potential. These are the response parameters which dictate how a soil will respond to applied stress. For example, since volumetric changes and, hence, liquefaction potential can be related to the distortional strain levels which a soil undergoes (Martin, Finn, and Seed, 1975), the shear stiffness or modulus of rigidity of a soil under a specific load level is of particular concern. Earthquake motions can be either amplified or attenuated, depending upon characteristics of the soil profile (and its interaction with the frequency content of the disturbing earthquake) which, in turn, depends upon the values of the stiffness and damping parameters involved.

Since many treatments of earthquake-induced liquefaction deal with vertically transmitted horizontal shear waves, one approach to analysis requires only a value for the shear modulus, G, together with a damping coefficient, to account for the energy absorption of the soil. Extensive experimental work dealing with these two parameters has been carried out by Seed and Idriss (1970), and Hardin and Drnevich (1970). These studies permit characterizing the shear response parameters of soil in terms of the basic soil index properties and the existing stress and strain states. For example, the shear modulus value for clean granular soils is related to void ratio, mean effective stress, maximum cyclic shear strain amplitude, and number of loading cycles (some soils have an additional dependency upon overconsolidation ratio, degree of saturation, and plasticity index). Soil damping, particularly in cohesionless soils, is at least partially due to relative movements between soil particles and, hence, is hysteric. The contribution by dry friction to the damping ratio should be substantially independent of strain rate. For analytical expediency damping is sometimes represented by an equivalent viscous damping. For soils, damping is generally specified as a percentage of critical damping, and measured in terms of specific damping capacity, related to the ratio of the area within a hysteric loop during a load cycle and the maximum stored energy during the cycle. Seed and Idriss (1970) have derived expressions for damping ratio as a function of strain level, number of cycles, frequency, mean effective stress, and the other index properties mentioned in reference to shear modulus.

The shear modulus is noted as increasing with density and confining pressure and decreasing with shear strain amplitude. Damping coefficients on the other hand increase with shear strain amplitude and appear to decrease with confining stress and increased density. Previous stress history is noted as increasing shear stiffness and decreasing damping. One application of the use of the foregoing soil parameters to earthquake response analysis has been incorporated into a computer program SHAKE (Schnabel, Lysmer and Seed, 1972) in which the shear modulus of granular materials is treated as:

$$G = A K_2 (\sigma)^a$$

Where A and a are constants, normally having values of 1,000 and 0.5, respectively, and K_2 is a function of the index properties of the soil and is an inverse function of the shear strain amplitude.

It has been found (Seed and Idriss, 1970; Hardin and Drnevich, 1970) that shear modulus values at any strain level may be normalized in terms of maximum shear modulus to permit a generalized relationship for many soil materials to be collapsed into a single relationship. Damping ratios, as mentioned, were found to vary as functions of soil index properties as well as the stress and strain states. Although cohesive materials have been treated in the same format as granular materials, their soil models, have not been found quite as satisfactory in this context. It is more expedient to normalize the shear modulus of clays in terms of the undrained shear strength, S_u, in the form of G/S_u versus shear strain amplitude. It is again possible to collapse the various shear modulus relationships into a single curve by normalizing them by the maximum way, modulus values determined at very small strain levels, such as by measuring shear wave velocities in the field, can be

used to predict the shear modulus under design loading conditions. Damping ratios for clays have been studied less extensively than for granular materials. Little data is available for materials other than sans and clays, but available information indicates that coarser grained materials such as gravels may be expected to behave as sands (Seed and Idriss, 1970; Hardin an Drnevich, 1970). Peats are generally treated in the same format as clays.

Standard Penetration Test (SPT) Correlations

The SPT blow count value, N, has been used as a means of estimating relative density and liquefaction. Early work related relative density to blow count as a function of overburden stress. Additional research showed other factors such as vertical stress, stress history and compressibility influenced results. Figure 1 shows the complexity of the relationships and is taken from work by Marcuson and Bieganousky (1977).

The energy efficiency of the drop hammer is an important factor affecting the SPT value. Typically, the average energy imparted by the falling weight is 60 percent of the theoretical " frictionless" value, although this can vary over a range of 30 to 90 percent. Additional variables affecting the N value are the type of hammer, the age of the rope, and the bore hole size. Correction factors are used to attempt to establish a constant energy ratio. The N_{60} is the N value corrected for the field procedures to an average energy ratio of 60 percent.

$$N_{60} = C_{ER}\ C_B\ C_S\ C_R\ N$$

where

C_{ER}	Energy ratio correction
C_B	Borehole diameter correction
C_S	Sampling method correction
C_R	Rod length correction
N	Measured SPT N value blows / foot

Table 2 shows the values for the correction factors. The energy efficiency depends on the size of the cathead and number of turns of rope. The standard practice in the United States uses two turns of rope on a large cathead.

The SPT N value varies with stress level; a correction factor is used for overburden stress.

where

$(N_1)_{60}$	N_{60} value corrected to a reference stress of one atmosphere
C_N	Correction factor for overburden stress

The C_N has been studied by several researchers and a range of relationships is presented in Figure 2 where

p_a	Atmospheric pressure (or 1 ton/sq. ft)
σ_{vo}	Vertical overburden effective stress

Figure 3 shows a data relating the relative density, D_r to the average particle size, D_{50} (particle size at which 50 percent of the sample is of finer size). The age of the deposit is also a factor affecting this relationship as is the overconsolidation ratio. The expression for relative density is:

$$D_r^2 = \frac{(N_1)_{60}}{C_P \, C_A \, C_{OCR}}$$

where

C_P	Factor for particle size
C_A	Factor for age
C_{OCR}	Factor for overconsolidation

The factors are given in Table 3.

Cone Penetration Test (CPT) Correlation

Work evolved using the CPT to predict relative density and liquefaction. Correction factors were developed for this test to standardize to reference conditions. The standardized cone tip resistance (q_n) is related to the measured cone tip resistance, q_c:

$$q_n = C_q \; q_c$$

where

C_q	Overburden stress correction factor

The value of C_q is essentially the same as the value of C_N used for the SPT and is

$$C_q = \frac{p_a}{(\sigma_{vo})^{0.5}}$$

Liquefaction Stress Ratio

Seed and Idriss (1971) have proposed a simple hand computation procedure for evaluating liquefaction. They assume that the shear stresses developed in a soil deposit are caused by upward-propagating shear waves. The depth to the soil region under liquefaction investigation is defined as h. The soil column above a depth h is assumed to behave as shown in Figure 4. The maximum shear stress at a depth h is related to the ground acceleration by:

$$\tau_{max} = \frac{\gamma h}{g} A_{max}\ r_d$$

where

γ	Total unit weight of soil
h	Depth to region where liquefaction is expected
A_{max}	Maximum surface acceleration
r_d	Acceleration correction factor

The factor r_d is used since the soil is a deformable body rather than a rigid one. Figure 4 gives a range of values for r_d with depth. Liquefaction is usually not experienced at depths greater than the upper 50 feet of soil. Since the actual time history of motion will have an irregular form, the average stress is arbitrarily taken as 65 percent of the maximum. Thus, the average stress τ_{av} is assumed to be:

$$\tau_{av} = 0.65\ \frac{A_{max}}{g}\ \gamma\, h\, r_d$$

The cyclic stress ratio is defined as:

$$\frac{\tau_{av}}{\sigma_{vo}'}$$

The above shows the computation of the cyclic stress ratio induced in the soil deposit at a given depth for a given earthquake level of seismic shaking. The strength of the soil and its capacity to withstand a given number of cycles of loading without liquefying can also be expressed in terms of the cyclic stress ratio. Liquefaction occurs when the demand exceeds the capacity.

The cyclic stress ratio capacity of a soil can be evaluated using a cyclic triaxial compression laboratory test or direct shear test. Alternatively the occurrence of

liquefaction can be predicted by use of a field test such as the SPT. The factor of safety is defined as:

$$FS = \frac{\text{Cyclic Stress Ratio to cause Liquefaction (CSRL)}}{\text{Cyclic Stress Ratio Induced by Earthquake (CSRE)}}$$

Liquefaction Predicted By Field Test

Seed and de Alba (1986) compiled occurrences of liquefaction and developed the relationship shown in Figure 5. The standard penetration test blow count data has been corrected for overburden stress and constant energy as noted above. They also developed a comparable relationship for the cone penetration test, Figure 6, which uses the cone tip resistance, q_n, corrected for the overburden stress:

$$q_n = q_c \, (p_a/\sigma_{vo})^{0.5}$$

Shibata and Teparaksa (1988) developed a relationship using cone penetration data obtained at earthquake sites in Japan, China and the United States. This relationship, Figure 7, is a further refinement.

Evaluation Of Liquefaction Potential

During a large earthquake, liquefaction poses a major threat to the Navy's waterfront structures. Therefore, assessment of earthquake hazards at waterfront sites must include an evaluation of liquefaction potential. The Naval Facilities Engineering command (NAVFAC) has developed a microcomputer program named LIQUFAC to evaluate soil liquefaction potential. LIQUFAC, short for Liquefaction Potential Analysis for Naval Facilities Sites, was developed to evaluate the safety factor against liquefaction for each soil layer during an earthquake, The program estimates the associated dynamic settlement. LIQUFAC deals with the case of level-ground conditions which assumes the soils are under zero driving shear stress.

The program was coded to use standard penetration test (SPT) or cone penetration test (CPT) data, soil properties, subsoil conditions, and earthquake characteristics. The program runs interactively on IBM PC compatible computers.

Input of earthquake characteristics requires the maximum horizontal ground acceleration, a_{max}, and the magnitude of the earthquake, M. A seismicity study plus soil data for the project site will usually provide adequate ground motion parameters for the analysis. The procedure uses data in Figure 6 to determine the factor of safety

The vertical deformation is computed using the dynamic soil properties of shear modulus or shear wave velocity and the Plasticity Index. Castro (1987) presents the

following procedures to estimate settlements as a result of earthquake loading for level ground sites which were used in the program.

1. Estimate the cyclic shear strains in the soil profile induced by the design earthquake. These can be computed using the equation, Dobry, et al. (1982):

$$\gamma_{cyc} = \frac{0.65\ a_{max}\ \sigma_{vo}\ r_d}{g\ G_{max}\ (G/G_{max})}$$

where:

γ_{cyc}	Cyclic shear strain.
g	Acceleration of gravity
G	Shear modulus of the soils.
G_{max}	Shear modulus of the soil at very small cyclic strains, 10^{-4} percent.
G/G_{max}	Effective modulus reduction factor for the soil, a function of the cyclic shear strain. The value of G/G_{max} is influenced by the Plasticity Index and is compatible with the computed cyclic shear strain.

2. Estimate the volumetric compression of each soil layer based on correlations between cyclic shear strain and volumetric strain. (LIQUFAC also allows for a user to define a correlation between volumetric strain and cyclic shear strain, if the user chooses to do so.)

3. Compute the volumetric compression in each soil layer by multiplying volumetric strains in each soil layer by the thickness of that layer. Obtain the earthquake induced settlement by summing the layer contributions.

The LIQUFAC program computes factor of safety against liquefaction and the induced settlement. The output includes distributions of the effective stress, the average corrected SPT values, and the average layer SPT value that will give a safety factor against liquefaction equal to 1. It also includes the distribution of dynamic soil properties, cyclic shear strains, volumetric strains, and one-dimensional settlement with depth. Figure 8 shows the graphic plot of an example run of the LIQUFAC program. NAVFAC uses this program to assess the liquefaction potential, and to determine the need for detailed dynamic soil property tests before designing waterfront structures.

Lateral Deformation And Spreading

The occurrence of liquefaction and its associated loss of soil strength can cause large horizontal deformations. These deformations are capable of causing failure of buildings, sever pipelines, buckle bridges, and topple retaining walls. The Navy sponsored

research to develop procedures for quantification of lateral deformation, Youd (1993). Three types of ground failure are possible. Flow failures may occur on steep slopes and lateral spread may occur on gentle slopes. A third type of failure involves ground oscillation on flat ground with liquefaction at depth decoupling surface layers. This decoupling allows rather large transient ground oscillations or ground waves.

Bartlett and Youd (1992) collected lateral spread case history data from eight earthquakes, six in western United States and two in Japan. The lateral spread data from the Japanese earthquakes are from a narrow range of seismic conditions, magnitude 7.5 and 7.7 earthquakes at source distances of 21 to 30 km. The six US earthquakes span a wider range of magnitudes (6.4 to 9.2) and greater range of source distances (up to 90 km), but all come from the western US, which is characterized by relatively high ground motion attenuation with distance from the seismic source. The observational data are primarily from stiff sites in regions of relatively high ground motion attenuation. Bartlett and Youd (1992) also compiled from published literature a lateral spread database consisting of 448 horizontal displacement vectors and 270 associated nearby bore-hole logs. A technique of stepwise multiple linear regression was applied to first define the factors that most influence ground displacement, and then to construct a regression model incorporating those factors.

Two statistically independent models were developed: a free-face model for areas near steep banks, and a ground-slope model for areas with gently sloping terrain. Several soil factors were tested in the models; those that were statistically significant are incorporated into the following equations.

For free-face conditions:

$$\begin{aligned} \mathrm{LOG}\,D_H = \; & -16.3658 + 1.1782\,M - 0.9275\,\mathrm{LOG}(R) - 0.0133\,R + 0.6572 \\ & \mathrm{LOG}\,(W) + 033483\,\mathrm{LOG}\,(T_{15}) + 4.5270\,\mathrm{LOG}\,(100 - F_{15}) \\ & - 0.9224\,D50_{15} \end{aligned}$$

For ground slope conditions:

$$\begin{aligned} \mathrm{LOG}\,D_H = \; & -15.7870 + 1.1782\,M - 0.9275\,\mathrm{LOG}\,(R) - 0.0133\,R + \\ & 0.4293\,\mathrm{LOG}\,(S) + 0.3483\,\mathrm{LOG}\,(T_{15}) + 4.5270\,\mathrm{LOG}\,(100 - F_{15}) \\ & - 0.9224\,D50_{15} \end{aligned}$$

Where:

- D_H Estimated lateral ground displacement in meters.
- $D50_{15}$ Average mean grain size in granular layers included in T_{15}, in millimeters.
- F_{15} Average fines content (fraction of sediment sample passing a No. 200 sieve) for granular layers included in T_{15}, in percent.

M Earthquake magnitude (moment magnitude).
R Horizontal distance from the seismic energy source, in kilometers.
S Ground slope, in percent.
T_{15} Cumulative thickness of saturated granular layers with corrected blow counts, $(N_1)_{60}$, less than 15, in meters.
W Ratio of the height (H) of the free face to the distance (L) from the base of the free face to the point in question, in percent.

To show the predictive performance of the above equations, Bartlett and Youd plotted predicted displacements against measured displacements recorded in the observational database (Figure 9). The solid diagonal line on the figure represents prefect prediction, i.e., predicted displacement equals measured displacement. The lower dashed line represents 100 percent over prediction, and the dashed upper line represents 50 percent under prediction. Approximately 90 percent of the data plot between these two dashed bounds. This grouping indicates that predicted displacements re generally valid within a factor of 2 and that doubling of the predicted displacement provides a displacement estimate with a high probability of not being exceeded.

The above equations are generally valid for stiff-soil sites in the Western US or within 30 km of the seismic source in Japan, i.e., the localities from which the case-history data were collected. The Navy used the equations as the basis for development of a computer program to allow rapid computation of a site.

Deformation of Nonliquefying Slopes

The effects of an earthquake on nonliquefying slopes can be determined using a conventional pseudostatic stability analysis in which the effects of the earthquake are represented as an equivalent static horizontal force acting on the slope and the factor of safety estimated. A factor of safety of less than 1.0 indicates the slope will yield and deformation can be expected. An approximate procedure for estimating the deformation uses the concept of a frictional sliding block or Newmark method. This procedure solves for the accumulated deformation of the sliding mass of soil by integrating increments of movement which occur each time an increment of ground acceleration exceeds the yield acceleration of the soil. The amount of permanent displacement depends on the maximum magnitude and duration of the earthquake. The ratio of maximum acceleration to yield acceleration of 2.0 will result in block displacements of the order of a few inches for a magnitude 6 1/2 earthquake and several feet for a magnitude 8 earthquake.

Significant pore pressure increases may be induced by earthquake loading in saturated silts and sands. For these soils a potential exists for a significant strength loss. For dense saturated sand, significant undrained shear strength can still be mobilized even when residual pore pressure is high. For loose sands, the residual undrained strength which can be mobilized after high pore pressure build-up is very low and is often less than the static undrained shear strength. This may result in flow slides or large ground deformations.

Criteria for Mapping Liquefaction Hazard Zones

California Department of Conservation, Division of Mines and Geology has established guidelines for mapping areas which might be susceptible to the occurrence of liquefaction. These zones establish where site-specific geotechnical investigations must be conducted to assess liquefaction potential and, if required, provide the technical basis to mitigate the liquefaction hazard. The following is taken directly from their criteria:

> Liquefaction Hazard Zones are areas meeting one or more of the following criteria:
>
> 1. Areas known to have experienced liquefaction during historic earthquakes. Field studies following past earthquakes indicate liquefaction tends to recur at many sites during successive earthquakes
>
> 2. All areas of uncompacted fills containing liquefaction susceptible material that are saturated, nearly saturated, or may be expected to become saturated.
>
> 3. Areas where sufficient existing geotechnical data and analyses indicate that the soils are potentially liquefiable. The vast majority of liquefaction hazard areas are underlain by recently deposited sand and/or silty sand. These deposits are not randomly distributed, but occur within a narrow range of sedimentary and hydrologic environments. Geologic criteria for assessing these environments are commonly used to delineate bounds of susceptibility zones evaluated from other criteria, such as geotechnical analysis (Youd, 1991) . Ground water data should be compiled from well logs and geotechnical borings. Analysis of aerial photographs of various vintages may delineate zones of flooding, sediment accumulation, or evidence of historic liquefaction. The Quaternary geology should be mapped and age estimates assigned based on ages reported in the literature, stratigraphic relationships and soil profile descriptions. In many areas of Holocene and Pleistocene deposition, geotechnical and hydrologic data are compiled. Geotechnical investigation reports with Standard Penetration Test (SPT) and/or Cone Penetration Test (CPT) and grain size distribution data can be used for liquefaction resistance evaluations.
>
> 4. Areas where geotechnical data are insufficient. The correlation of Seed et al. (1985) , and the $(N_1)_{60}$ data can be used to assess liquefaction susceptibility. Since geotechnical analyses are usually made using limited available data the susceptibly zones should be delineated by use of geologic criteria. Geologic cross sections, tied to boreholes and/or trenches, should be constructed for correlation purposes. The units characterized by geotechnical analyses are correlated with surface and subsurface units and extrapolated for the mapping project.

CDMG criteria uses the minimum level of seismic excitation for liquefaction hazard zones to be that level defined by a magnitude 7.5-weighted peak ground surface acceleration for UBC S2 soil conditions with a 10 percent probability of exceedance over a 50-year period.

In areas of limited or no geotechnical data, susceptibility zones are identified by CDMG geologic criteria as follows:

> (a) Areas containing soil deposits of late Holocene age (current river channels and their historic floodplains, marshes and estuaries), where the magnitude 7.5-weighted peak acceleration that has a 10 percent probability of being exceeded in 50 years is greater than or equal to 0.10 g and the water table is less than 40 feet below the ground surface; or
>
> (b) Areas containing soil deposits of Holocene age (less than 11,000 years), where the magnitude 7.5-weighted peak acceleration that has a 10 percent probability of being exceeded in 50 years is greater than or equal to 0.20 g and the historic high water table is less than or equal to 30 feet below the ground surface; or
>
> (c) Areas containing soil deposits of latest Pleistocene age (between 11,000 years and 15,000 years), where the magnitude 7.5-weighted peak acceleration that has a 10 percent probability of being exceeded in 50 years is greater than or equal to 0.30 g and the historic high water table is less than or equal to 20 feet below the ground surface.

According to CDMG, the Quaternary geology may be taken from existing maps, and hydrologic data should be compiled. Application of this criteria permits development of liquefaction hazard maps which definite regions requiring detailed investigation, allowing concentration of sampling and testing in areas requiring most delineation.

Code Provisions and Factors Of Safety Against Liquefaction

In general building codes do not give extensive guidance for liquefaction apart for the need for investigating a site for geologic hazards. The AASHTO Standard Specification For Highway Bridges (1992) suggests the factor of safety of 1.5 is desirable to establish a reasonable measure of safety against liquefaction in cases of important bridge sites. While not specifically stated it is presumed that this is to be used in conjunction with their acceleration maps which give a 10 percent probability of exceedance in 50 years.

Response of Anchored Sheet Pile Walls

There is extensive experience on the performance of anchored sheetpile walls. Large scale liquefaction of loose saturated cohesionless soils in the backfill have caused major failures. Typical failures take the form of excessive permanent seaward tilting with associated movement of the anchor block. Associated with this is the settlement and cracking of the backfill soil. Gazetas and Dakoulas (1991) review procedures used to analyze quaywalls. Pseudostatic procedures are used to determine lateral earth pressures after the well known Mononobe-Okabe approach. Statistics show that performance of quaywalls over the last 45 years has not improved despite increases in the seismic coefficients and refinements in the design methods. The dominant factor in failures of these walls is the loss of strength of the backfill and foundation soils. The pseudo static method of analysis suffers from three significant deficiencies: the failure to account for the loss of strength associated with the generation of excess pore pressure, the overestimation of the passive soil resistance of the anchor, and the inability to include the deformation and movement of the wall and soil. Many designs underestimated the level of seismic exposure and the design procedure ignores the vertical component of acceleration. which can increase the effective acceleration relating to active and passive earth pressures. Gazetas and Dakoulas (1991) develops an empirical design chart based on numerous case studies of sheetpile walls which can be used to enhance conventional pseudostatic procedures. A horizontal acceleration factor is defines as:

$$k_h = \frac{2a_h}{3g}$$

A vertical acceleration factor may be assumed as two-thirds of the horizontal.

$$k_v = 2/3\ (k_h)$$

An effective acceleration coefficient is defined as:

$$k_e = \frac{k_h}{1 - k_v}$$

For cohesionless soils under water, the value of k_e may be increased by 1.5 to account for the potential of strength degradation from porewater pressure buildup. Figure 9 shows the nomenclature used. Figure 10 shows relationships for the active failure surface inclination, α_{ae}, and the active and passive seismic pressure coefficients as functions of the effective acceleration. The effective anchor distance, EAI is defined as

$$EAI = d / H$$

Having the effective acceleration coefficient one may determine the failure surface inclination and the seismic pressure coefficients. A trial value of EAI may be selected and the anchor length determined using Figure 11 and:

$$EPI \approx \frac{K_{PE}}{K_{AE}} (r^2 \ (r+1))$$

where

$$r = f / (f + H)$$

$$L \geq (H + f) \operatorname{Cot} (\alpha_{ac}) + (EAL_c) \ H$$

The above establishes a minimum anchorage length for safe performance based on field observations of damaged structures.

Observations of wall displacement have been correlated to damage and failure; these values can serve as criteria for limiting deflection to achieve performance.

Description of Damage	Permanent Displacement at Top of Sheetpile (inches)
No damage	1
Negligible damage to the wall but noticeable damage to appurtenant structures	4
Noticeable damage to wall	12
General shape of anchored sheetpile preserved, but significantly damaged	24
Complete destruction, no recognizable shape of wall	48

Liquefaction Remediation

Liquefaction remediation must address the specifics of the problem on a case by case basis. These specifics include the local site conditions, the type of structure, and the potential for flows and settlements. When liquefaction occurs, there can be a potential for extensive lateral flow slides which can affect a large area, a global site instability. Also there can be local soil settlements and bearing failures which affect a structure on a local level. Specific types of structures can have specific associated problems. Buried structures can become buoyant. Retaining structures where the backfill has liquefied can experience increased lateral loading and deformation. Potential suitable methods of remediation include:

- removal of liquefiable material and replacement
- site dewatering or improved drainage such as stone columns
- insitu site improvement
- containment or encapsulation
- modification of structure geometry
- deep foundations such as piles
- alternative site selection

Several methods of densification have been used including vibroprobe, vibro-compaction, dynamic compaction, compaction grouting, and compaction piles. Substitution or replacement of soil to improve drainage has been used including vibro-replacement and stone columns. Techniques like stone columns achieve their effectiveness by replacing liquefiable cohesionless soils with stiffer columns of gravel and rock which improves strength and promotes drainage. Cement grouting, jet grouting and deep mixing have been used as chemical means of eliminating/reducing liquefaction potential. Surcharging a site increases liquefaction resistance by increasing the effective confining pressures. Table 4 presents a summary of methods used for remediation and their relative cost as reported by Professor Whitman (NRC 1985). Navy facilities on Treasure Island during the 1989 Loma Prieta earthquake can attest to the effectiveness of remediation. Areas where remediation was done performed well while other areas suffered settlements of 6 to 8 inches and lateral spreads. Observation of damage during the 1995 Hyogoken Nanbu (Kobe) Earthquake again confirmed the performance of improved sites. Preloading, sand drains, sand compaction piles, and vibro-compaction were shown to be effective.

Method	Vertical Settlement	
	Range (cm)	Average (cm)
Untreated	25 to 95	42
Preloading	15 to 60	30
Sand drains	0 to 40	15
Sand drains & preloading	0 to 25	12
Vibro-compaction	0 to 5	near 0
Sand compaction piles	0 to 5	near 0

Generally costs increase from dynamic compaction to vibro-compaction to replacement The measure of effectiveness of a remediation undertaking is the increase in minimum soil density and specifications usually measure this by the improvement in penetration resistance or laboratory testing. Engineering practice tends to be conservative and factors of safety from 1.5 to 2.0 against liquefaction are often specified. These values may be harder to achieve at the waterfront in regions of high seismicity.

Example Calculation of Liquefaction

This example is taken directly from NCEL Report N 1862 by Youd (1993).

(1) The first step in a calculation of liquefaction susceptibility is to define the design peak horizontal acceleration, A_{max}, and earthquake magnitude, M. Procedures for making these estimates are given NFESC TR 2016. For this example, we shall assume a design earthquake with a magnitude of 6.5 that produces a peak acceleration of 0.30 g at the site in question.

(2) The second step is to develop a characteristic soil profile for the locality to be evaluated. Procedures such as those outlined in NAVFAC DM-7.1 Chapters 2 (Field Exploration Testing and Instrumentation) and 3 (Laboratory Testing) should be used to delineate and define soil stratigraphy. These manuals also provide suggested procedures for drilling and retrieving samples and for conducting of classification and index tests. For this example calculation, we shall assume the soil profile shown in Figure 12 and the soil properties listed in Table 5 are representative of the site.

(3) The third step is to calculate the cyclic stress ratio generated by the earthquake (CSRE) at each depth in question. For example, CSRE might be calculated at the depth of each standard penetration test or it might be calculated and plotted as a continuous curve versus depth. Several computer programs are available that facilitate these calculations. For this example, however, we will follow a step by step procedure to illustrate the calculation of CSRE at a depth of 15 feet.

$$\text{CSRE} = \tau_{av}/\sigma_{vo}' = 0.65(A_{max} / g)(\sigma_{vo} / \sigma_{vo}')\, r_d$$

The total overburden pressure, σ_{vo} at a depth of 15 ft for this example is:

$$= (110 \text{ lb/ft}^3)(5.0 \text{ ft}) + (120 \text{ lb/ft}^3)(10.5 \text{ ft}) = 1{,}750 \text{ lb/ft}^3$$

The effective overburden pressure, σ_{vo}' for this example is:

$$\sigma_{vo}' = \sigma_{vo} - u = 1{,}750 \text{ lb/ft}^2 - (15 \text{ ft} - 4 \text{ ft})(62.4 \text{ lb/ft}^3) = 1{,}064 \text{ lb/ft}^2$$

where u is pore-water pressure. From Figure 4, the coefficient r_d at a depth of 15 ft is 0.97. Applying these values yields the following CSRE:

$$\text{CSRE} = \tau_{av}/\sigma_{vo}' = (0.65)(0.30g)(1{,}750 \text{ lb/ft}^2/1{,}064 \text{lb/ft}^2)(0.97) = 0.31$$

(4) The fourth step is to calculate the cyclic stress ratio required to cause liquefaction (CSRL). That ratio is determined by correlation with $(N_1)_{60}$ through the curves drawn on Figure 5.

$$(N_1)_{60} = C_n\,(ER/60)\,N$$

For this example, we assume an energy ratio for the standard penetration hammer used in the field SPT test was measured at 50%. C_n may he determined directly from Figure 13. For an effective overburden pressure of 1,064 lb/ft^2 (1.06 Kip/ft^2), C_n = 1.37. Thus,

$$(N_1)_{60} = (1.37)(50/60)(12 \text{ blows/ft}) = 13.6 \text{ blows/ft}$$

This value along with all the other calculated $(N_1)_{60}$ values for this example soil profile are listed in Table 5 and plotted on Figure 12.

From the curves in Figure 4, an $(N_1)_{60}$ of 13.6 yields a CSRL of 0.17, which is the minimum CSRL that is required to generate liquefaction for a magnitude 7.5 earthquake. To correct the CSRL to a magnitude 6.5 earthquake, the CSRL of 0.17 must be multiplied by the appropriate magnitude scaling factor interpolated from Table 6. For a magnitude 6.5 earthquake, that factor is 1.19. Thus, the minimal CSRL required to cause liquefaction at a 15-ft depth in the given soil profile is:

$$\text{CSRL} = (1.19)(.17) = 0.20$$

(5) The factor of safety against liquefaction at a depth of 15 ft in the soil profile is calculated:

$$\text{FS} = \text{CSRL/CSRE} = 0.20/0.31 = 0.65$$

Thus, liquefaction would be expected to readily develop at a depth of 15 ft for the given design earthquake and site conditions. Factors of safety calculated for the depth of each standard penetration test in the soil profile are listed in Table 5 and plotted on Figure 12.

(6) As noted above, for soils containing more than 35% fines the curves in Figure 5 may be used as a conservative estimate for liquefaction hazard, provided that all of the following criteria suggested by Seed and others (1983) are met:

- The weight of soil particles finer than 0.005 mm (clay-size particles) is less than 15% of the dry weight of a specimen of the soil.
- The liquid limit of soil is less than 35%.
- The moisture content of the in-place soil is greater than 0.9 times the liquid limit.

For example, consider the soil in the silty clay layer at a depth of 21 ft to 24 ft as shown on Figure 5. The silt in that layer has a clay content of 13%, a liquid limit of 31% and a plastic limit of 22%, and a natural moisture content of 26%.

- By criterion I, the soil clay content of 13% is less than 15%, which does not exclude liquefaction.
- By criterion 2, the liquid limit of 32% is less than 35%, which does not exclude liquefaction.

- By criterion 3, the moisture content of the soil of 26% is less than 0.9 times the liquid limit of 32% (i.e. 0.9 X 32% = 29% >26%), which excludes liquefaction because the silt is over consolidated and thus immune to liquefaction.

Because the sediment in question does not meet all three of the criteria, the layer is classed as nonliquefiable

To illustrate the calculation of lateral ground displacement using the relationship developed by Professor Youd (1993), the example problem shown above is continued. This work is taken directly from work by Youd developed for the Navy and presented in NCEL TN-1862. Consider the hypothetical soil stratigraphy and ground conditions shown on Figure 14. Soil properties for the various layers are listed in Table 5 and plotted on Figure 12. From the cross section of the site, the height of the free face (channel depth) is noted as 16 ft. The planned structure is located 150 ft from the base of the free face. Thus,

$$W = (16\ \text{ft}/150\ \text{ft})(100) = 10.7\%.$$

The gentle ground slope of the terrain at the tower site has is characterized by a rise of elevation of 1.0 ft over a distance of 200 ft yielding a ground slope, S, of *0.5%.*

From a review of Figures 12 and 14 and the soil-property data in Table 5, the liquefiable layer is divided into two sublayers: Layer 1 is composed of sand to silty sand with a thickness, T_{15}, of 12 ft (3.6 m), an average fines content, F_{15}, of 6.5%, and an average mean-grain size, $D50_{15}$, of 0.405 mm. Layer 2 is composed of silty sand with a T_{15} of 3 ft (0.9 m), F_{15} of 43, and $D50_{15}$ of 0.11 mm. Application of those parametric values yields the following results:

For free-face conditions:

For layer 1,

$$\begin{aligned}\text{Log } D_{H1} = &-16.366 + 1.1782\,(6.5) - 0.9275\ \text{Log}(11\ \text{km}) - 0.0133(11\ \text{km}) + \\ &0.6572\ \text{Log}(10.7\%) + 0.3483\ \text{Log}(3.7\ \text{m}) + \\ &4.527\ \text{Log}(100 - 6.5\%) - 0.9224\,(0.405\ \text{mm})\end{aligned}$$

$$= -0.3972$$

and, $D_{H1} = 0.40$ m (1.34 ft)

For layer 2,

$$\begin{aligned}\text{Log } D_{H2} = &-16.366 + 1.1782(6.5) - 0.9275\ \text{Log}(11\ \text{km}) - 0.0133(11\ \text{km}) + \\ &0.6572\ \text{Log}(10.7\%) + 0.3483\ \text{Log}(0.9\ \text{m}) + 4.527\ \text{Log}(100 - 43\%) - \\ &0.9224\,(0.11\ \text{mm})\end{aligned}$$

$$= -1.3119$$

and, $D_{H2} = 0.05$ m (0.16 ft)

The total free-face displacement is the sum of the component displacements:

$$D_H = 0.40\text{m} + 0.05\ \text{m} = 0.45\ \text{m}\ (1.50\ \text{ft})$$

For ground slope conditions:

For Layer 1,

$$\text{Log } D_{H1} = -15.787 + 1.1782\,(6.5) - 0.9275\,\text{Log}(11\text{ km}) - 0.0133(11\text{ km}) + 0.4293\,\text{Log}(0.5\%) + 0.3483\,\text{Log}(3.7\text{ in}) + 4.527\,\text{Log}(100 - 6.5\%) - 0.9224\,(0.405\text{ mm}) - 0.6239$$

and, $D_{H1} = 0.24$ m (0.80 ft)

For layer 2,

$$\text{Log } D_{H2} = -15.787 + 1.1782\,(6.5) - 0.9275\,\text{Log}(11\text{ km}) - 0.0133(11\text{ km}) + 0.4293\,\text{Log}(0.5\ \%) + 0.3483\,\text{Log}(0.9\text{ m}) + 4.527\,\text{Log}(100 - 43\%) - 0.9224\,(0.11\text{ mm}) -1.5387$$

and, $D_{H2} = 0.03$ m (0.10 ft)

The total ground slope displacement is the sum of the component displacements:

$$D_H = 0.24\text{m} + 0.03\text{ m} = 0.27\text{m}\ \ (0.90\text{ ft})$$

Only the larger of the two estimated displacements need be used in the design analysis. In this instance that displacement is 1.5 ft. (If the designer wished to be ultraconservative, the displacements predicted for ground-slope conditions could be added to the free-face displacement. That degree of conservatism, however, is not required.) Doubling of the displacement predicted yields a value with a high probability of not being exceeded. In this instance the predicted displacement of 1.5 ft should be doubled to 3.0 ft for conservative design associated with essential construction

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Table 1
Unified Classification Of Soil.

Major Divisions			Letter Symbol	Typical Descriptions	Probability of Liquefaction
Coarse-grained soils More than 50% of material is larger than no. 200 sieve size	Gravel and Gravelly soils More than 50% of coarse fraction retained on no. 4 sieve	clean gravels (little or no fines)	GW	well-graded gravels, gravel-sand mixtures, little or no fines	low to moderate
			GP	poorly-graded gravels, gravel-sand mixtures, little or no fines	low
		gravels with fines (appreciable amount of fines)	GM	silty gravels, gravel-sand-silt mixtures	moderate
			GC	clayey gravels, gravel-sand-clay mixtures	low
	Sand and Sandy soils More than 50% of coarse fraction passing no. 4 sieve	clean sand (little or no fines)	SW	well-graded sands, gravelly sands, little or no fines	high
			SP	poorly-graded sands, gravelly sands, little or no fines	moderate to high
		sands with fines (appreciable amount of fines)	SM	silty sands, sand-silt mixtures	moderate to high
			SC	clayey sands, sand-clay mixtures	low to moderate
Fine-grained soils More than 50% of material is smaller than no. 200 sieve size	Silts and Clays	liquid limit greater than 50	ML	inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	low to high
			CL	inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	NA
			OL	organic silts and organic silty clays of low plasticity	NA
	Silts and Clays	liquid limit greater than 50	MH	inorganic silts, micaceous or diatomaceous fine sand or silty soils	NA
			CH	inorganic clays or high plasticity, fat clays	NA
			OH	organic clays of medium to high plasticity, organic silts	NA
Highly Organic Soils			PT	peat, humus, swamp soils with high organic contents	NA

Table 2

Spt Correction Factors For Field Procedures

FACTOR	EQUIPMENT VARIABLES	CORRECTION TERM	VALUE
Energy ratio	Safety hammer Donut hammer	C_{ER}	0.9 0.75
Borehole diameter	65 to 115 mm (2.5 to 4.5 in) 150 mm (6 in) 200 mm (8 in)	C_B	1.0 1.05 1.15
Sampling method	Standard Sampler Sampler without liner	C_S	1.0 1.2
Rod length	> 10 m (> 30 ft) 6 to 10 m (20 to 30 ft) 4 to 6 m (13 to 20 ft) 3 to 4 m (10 to 13 ft)	C_R	1.0 0.95 0.85 0.75

Source: Based on Skempton (1986)

Table 3

Spt Correction Factors For Sand Variables

EFFECT	PARAMETER	CORRECTION TERM	CORRECTION VALUE
Particle size	D_{50} of sand	C_P	$60 + 25 \log D_{50}$ (D_{50} in mm)
Aging	Time (t)	C_A	$1.2 + 0.05 \log (t/100)$
Overconsolidation	$OCR = \sigma_p/\sigma_{vo}$	C_{OCR}	$OCR^{0.18}$

Table 4
Liquefaction Remediation Measures

Method	Principle	Most Suitable Soil Conditions/Types	Maximum Effective Treatment Depth	Relative costs
(1) Blasting	Shock waves and vibrations cause limited liquefaction , displacement, remolding, and settlement to higher density.	Saturated, clean sands; partly saturated sands and silts after flooding.	>40 m	Low
(2) Vibratory probe (a) Terraprobe (b) Vibrorods (c) Vibrowing	Densification by vibration; liquefaction-induced settlement and settlement in dry soil under overburden to produce a higher density.	Saturated or dry clean sand; sand.	20 m routinely (ineffective above 3-4 m depth); >30 m sometimes; vibrowing, 40 m	Moderate
(3) Vibrocompaction (a) Vibroflot (b) Vibro-Composer System	Densification by vibration and compaction of backfill material of sand or gravel.	Cohesionless soils with less tan 20% fines.	>20m	Low to moderate
(4) Compaction piles	Densification by displacement of pile volume and by vibration during driving, increase in lateral effective earth pressure.	Loose sandy soil; partly saturated clayey soil; loess.	>20 m	Moderate to high
(5) Heavy tamping (dynamic compaction	Repeated application of high-intensity impacts at surface.	Cohesionless soils best, other types can also be improve.	30 m (possibly deeper)	Low
(6) Displacement/ compaction) grout	Highly viscous grout acts as radial hydraulic jack when pumped in under high pressure.	All soils.	Unlimited	Low to moderate

(7) Surcharge/buttress	The weight of a surcharge/buttress increases the liquefaction resistance by increasing the effective confining pressures in the foundation.	Can be placed on any soil surface.	—	Moderate if vertical drains used
(8) Drains (a) Gravel (b) Sand (c) Wick (d) Wells (for permanent dewatering	Relief of excess pore water pressure to prevent liquefaction. (Wick drains have comparable permeability to sand drains). Primarily gravel drains; sand/wick may supplement gravel drain or relieve existing excess pore water pressure. Permanent dewatering with pumps.	Sand, silt, clay	Gravel and sand>30 m; depth limited by vibratory equipment; wick, >45 m	Moderate to high
(9) Particulate grouting	Penetration grouting-fill soil pores with soil, cement, and/or clay.	Medium to coarse sand and gravel.	Unlimited	Lowest of grout methods
(10) Chemical grouting	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate.	Medium silts and coarser.	Unlimited	High
(11) Pressure injected lime	Penetration grouting-fill soil pores with lime.	Medium to coarse sand and gravel.	Unlimited	Low
(12) Electrokinetic injection	Stabilizing chemicals moved into and fills soil pores by electro-osmosis or colloids into pores by electrophoresis.	Saturated sands, silts, silty clays.	Unknown	Expensive

(13) Jet grouting	High-speed jets at depth excavate, inject, and mix a stabilizer with soil to form columns or panels.	Sands, silts, clays.	Unknown	High
(14) Mix-in-place piles and walls	Lime, cement, or asphalt introduced through rotating auger or special inplace mixer.	Sand, silts, clays, all soft or loose inorganic soils.	>20 m (60 in obtained in Japan)	High
(15) In-situ vitrification	Melts soil in place to create an obsidian-like vitreous material.	All soils and rock.	>30 m	Moderate
(16) Vibro-replacement stone and sand columns (a) Grouted (b) Not grouted	Hole jetted into fine-grained soil and backfilled with densely compacted gravel or sand hole formed in cohesionless soils by vibro techniques and compaction of backfilled gravel or sand. For grouted columns, voids filled with a grout.	Sands, silts, clays.	>30 m (limited by vibratory equipment)	Moderate
(17) Root piles, soil nailing	Small-diameter inclusions used to carry tension, shear, compression.	All soils.	Unknown	Moderate to high

Table 5

Boring Log Soil Profile

Depth ft	Soil Description	N_m blow/ft	$(N_1)_{60}$ blow/ft	Fines %	Clay %	D_{50} mm	Unit Weight lb/ft^3	CSRL	CSRE	Factor of Safety
3	Silty clay (CL)	NA		87	43		110	NA	0.19	NA
6	Sand (SW)	6	8.5 ?	3	0	0.43	120	0.12	0.24	0.50
9	Sand (SW)	5	6.2 ?	5	0	0.51	120	0.07	0.27	0.26
12	Sand with silt (SW-SM)	15	18.6	10	0	0.31	120	0.30	0.29	1.02
15	Sand with silt (SW-SM)	12	13.6	8	0	0.37	120	0.20	0.31	0.63
18	Silty sand (SM)	9	9.4	43	2	0.11	120	0.21	0.32	0.66
21	Silt (ML)	9	8.8	88	13	0.03	120	NA	0.32	NA
24	Silty sand (SM)	17	15.9	21	0	0.22	120	0.30	0.33	0.92
27	Silty sand (SM)	18	15.9	30	1	0.20	120	0.33	0.33	1.00
30	Silty sand (SM)	21	17.7	37	1	0.18	120	0.47	0.33	1.42
33	Silty sand (SM)	31	25.1	35	0	0.25	120	>1	0.33	>2
36	Silty sand (SM)	33	25.6	28	0	0.23	120	>1	0.32	>2
39	Silty sand (SM)	32	24.0	18	0	0.30	120	>1	0.31	>2
42	Clay (ML)									

Table 6

Scaling Factors For Cyclic Stress Ratio

Earthquake Magnitude (M)	Number of Representative Cycles at 0.65 τ_{max}	Factor to Correct Abscissa of Curve in Figure 5
8.5	26	0.89
7.5	15	1.0
6.75	10	1.13
6.0	5-6	1.32
5.25	2-3	1.5

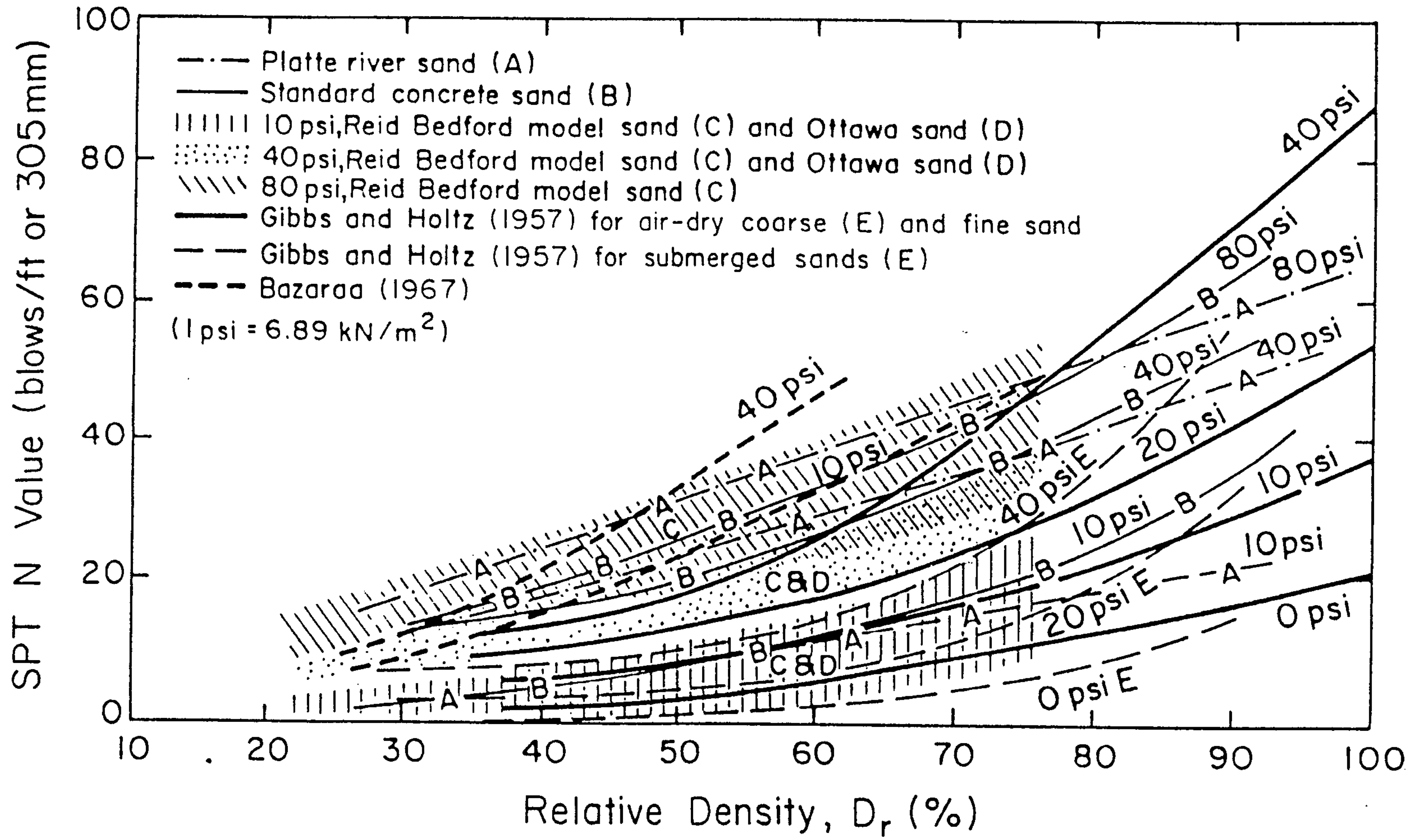

Figure 1 . Relative density as a function of blowcount for several sands. from Marcuson and Bieganousky (1977)

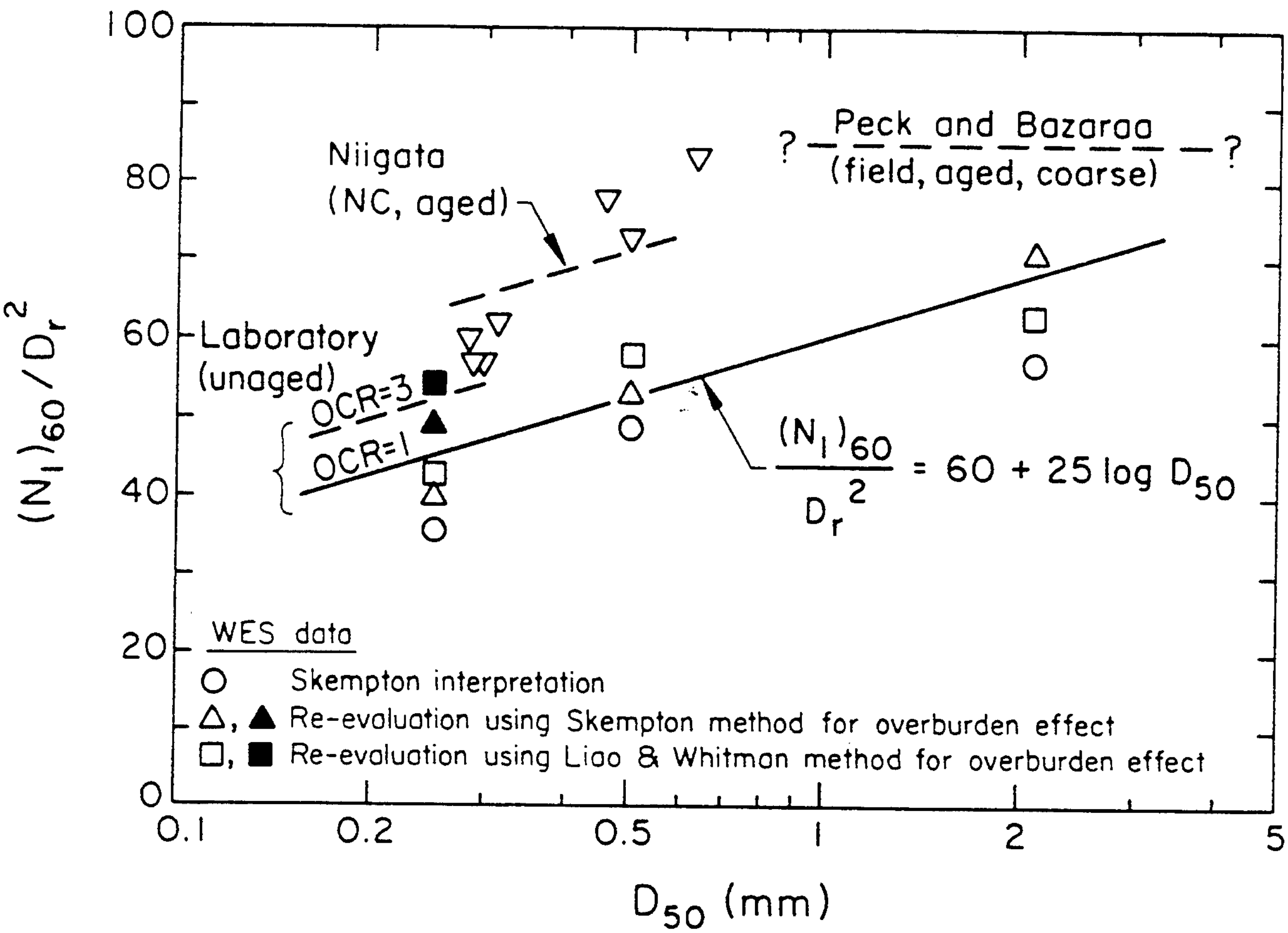

Figure 3. Particle size effect on blow count for sands.
from Kulhawy and Mayne (1990), EPRI EL6800

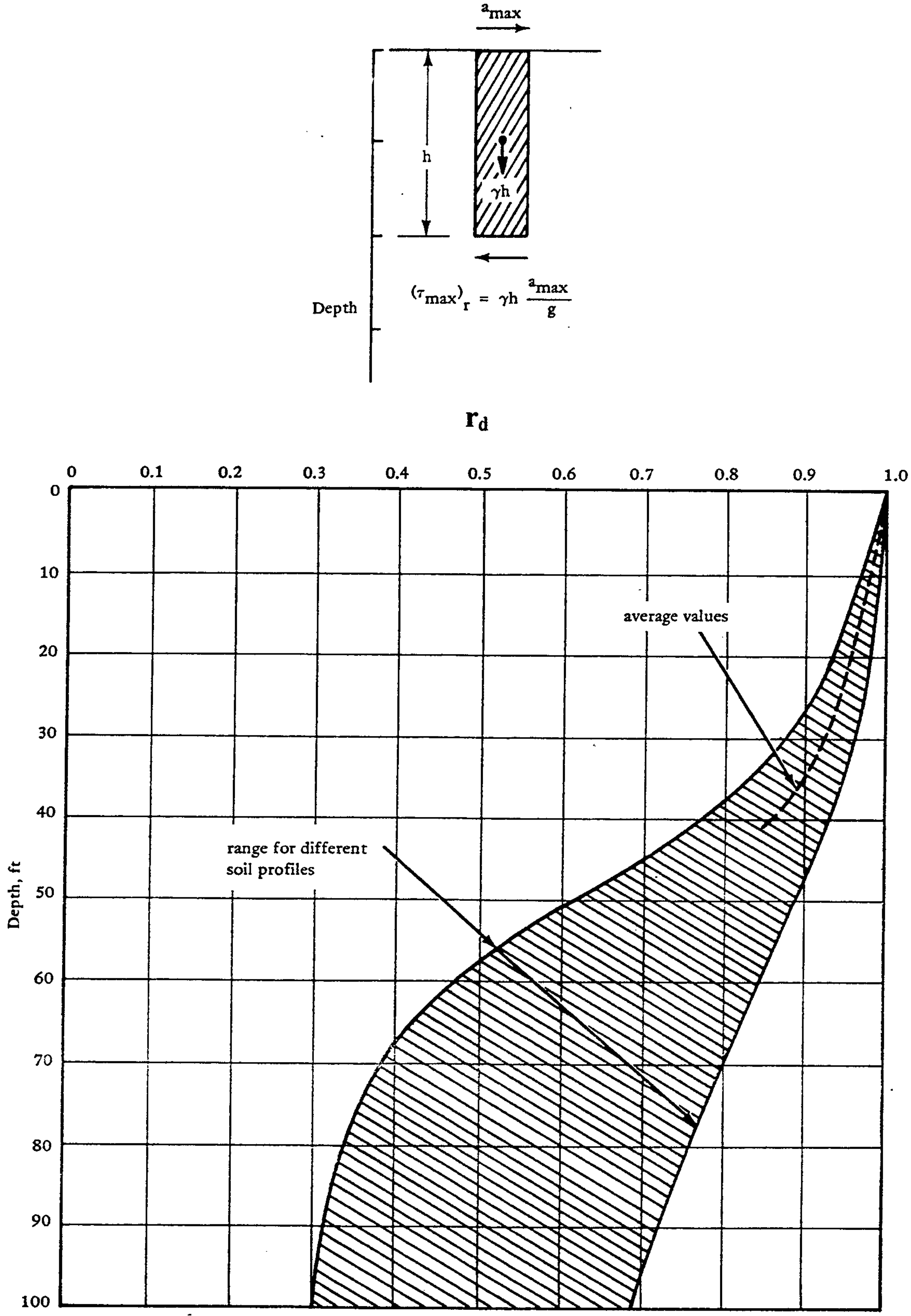

Figure 4. Range of values for r_d.

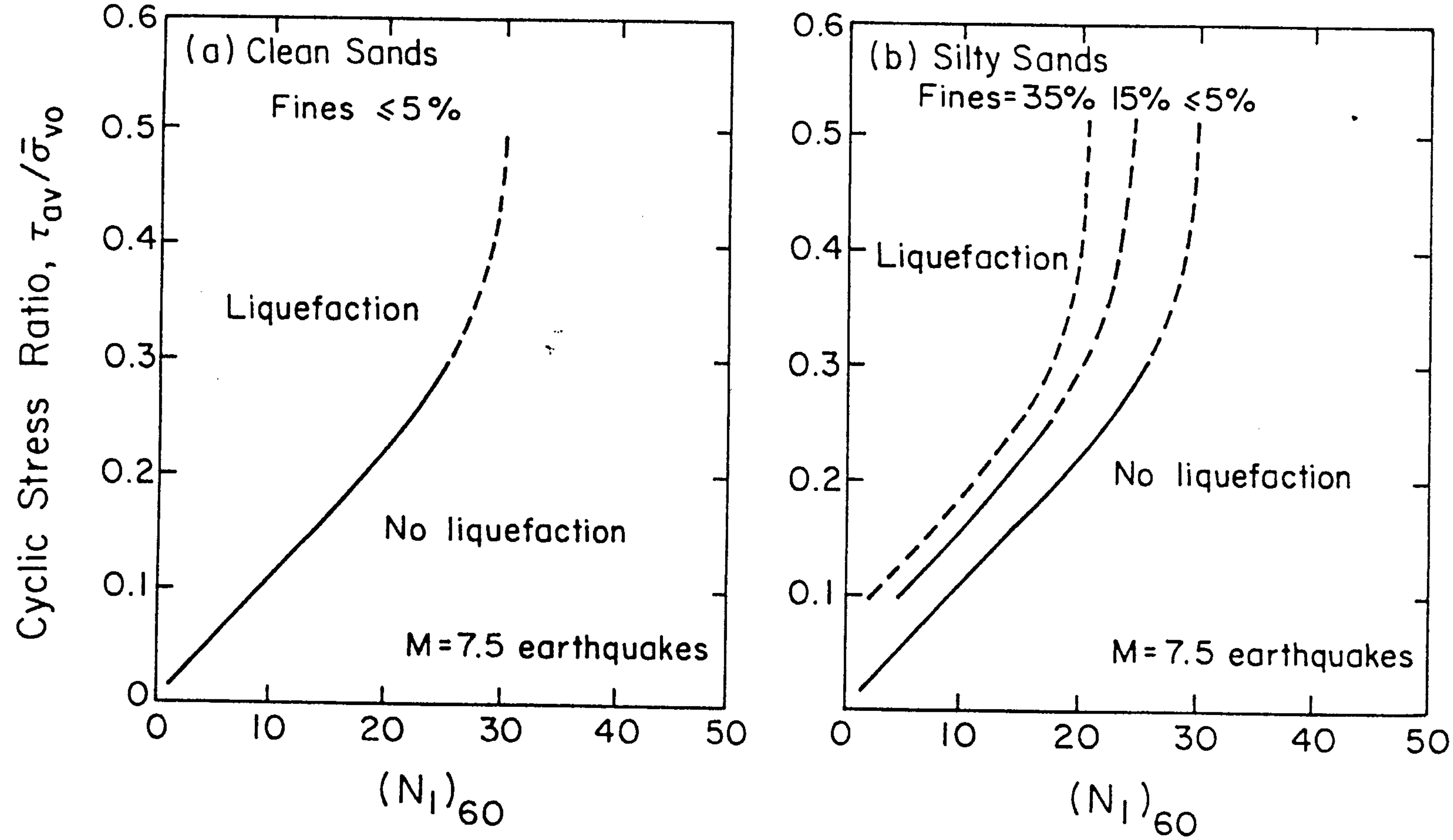

Figure 5. Liquefaction resistance with corrected blow count. from Seed and de Alba (1986)

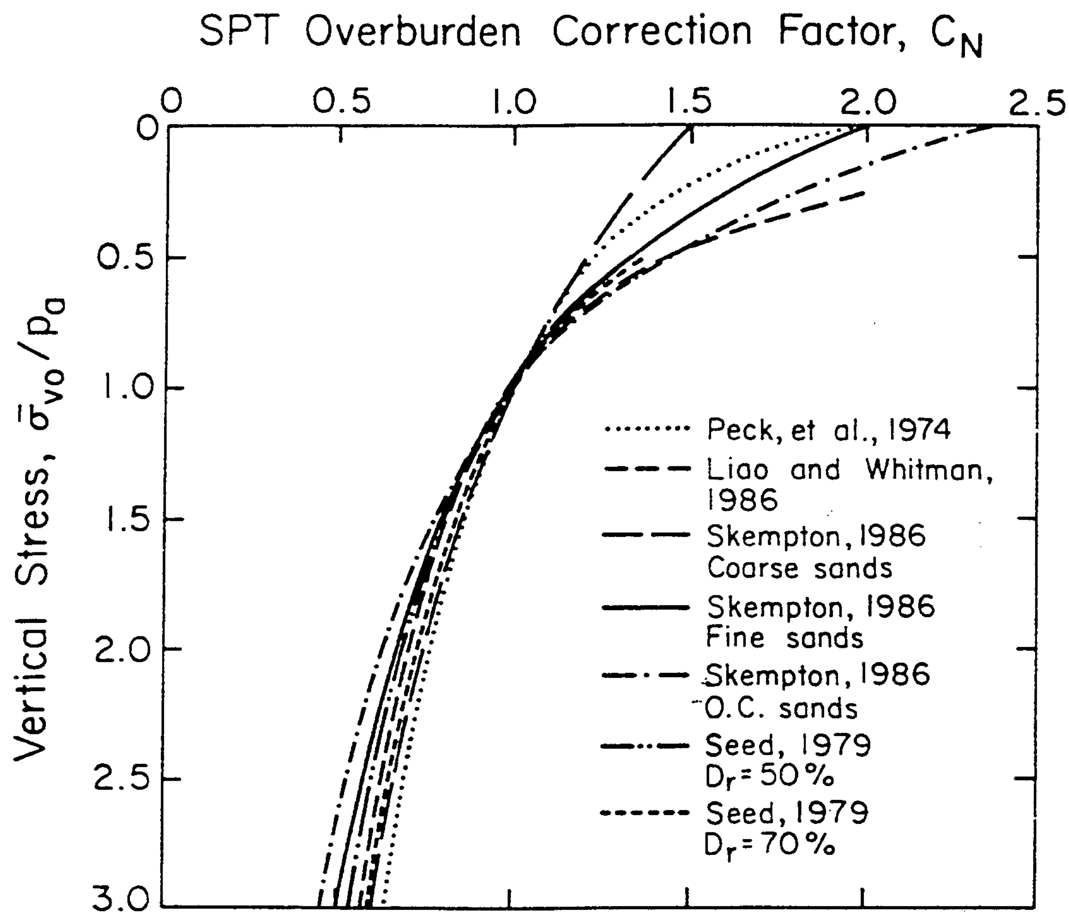

Figure 2. Comparison of SPT correlations. from Kulhawy and Mayne (1990), EPRI EL6800

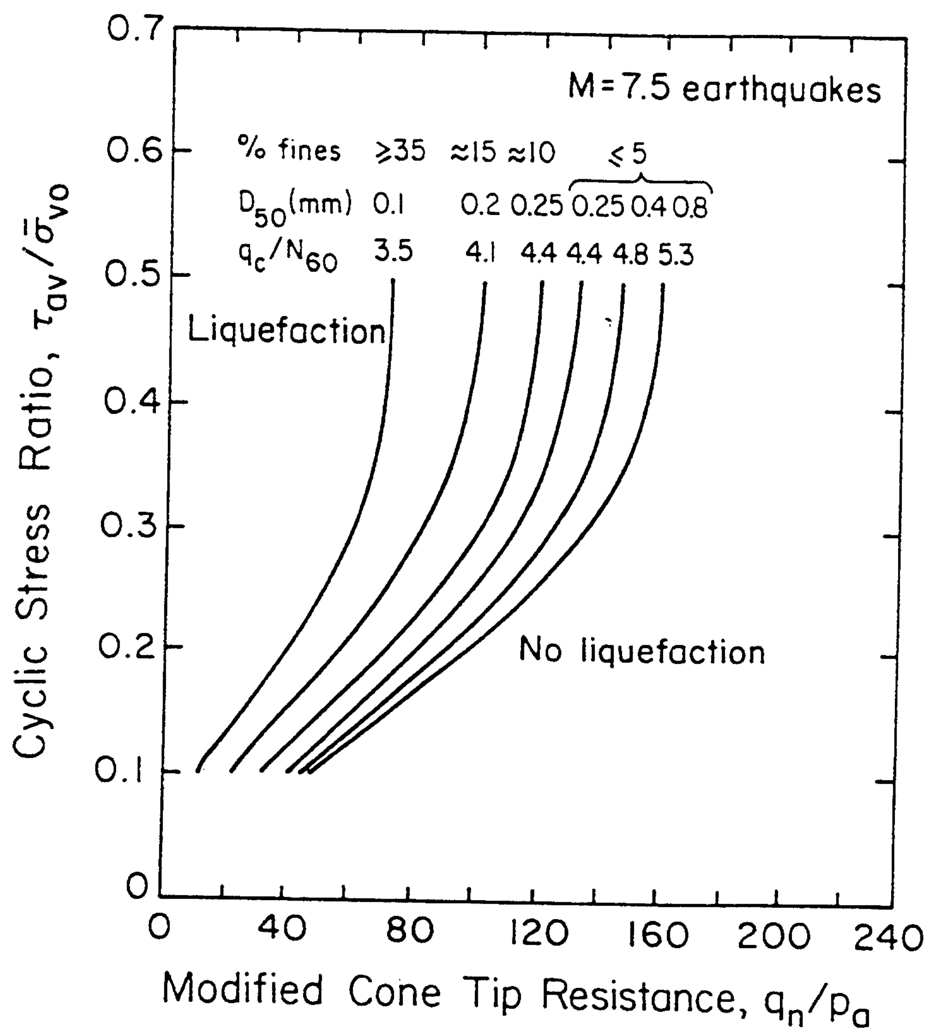

Figure 6. Liquefaction resistance with CPT values. from Seed and de Alba (1986)

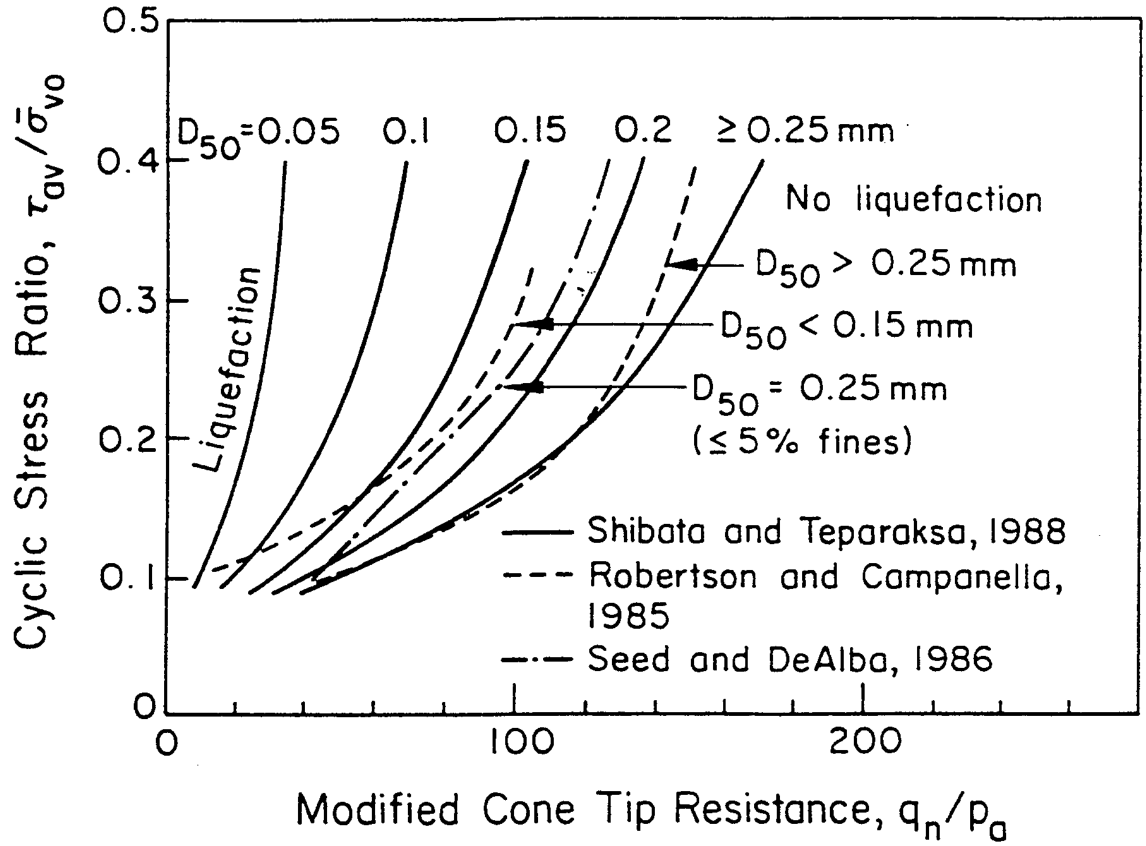

Figure 7. Liquefaction resistance with CPT values. from Shibata and Teparaksa (1988)

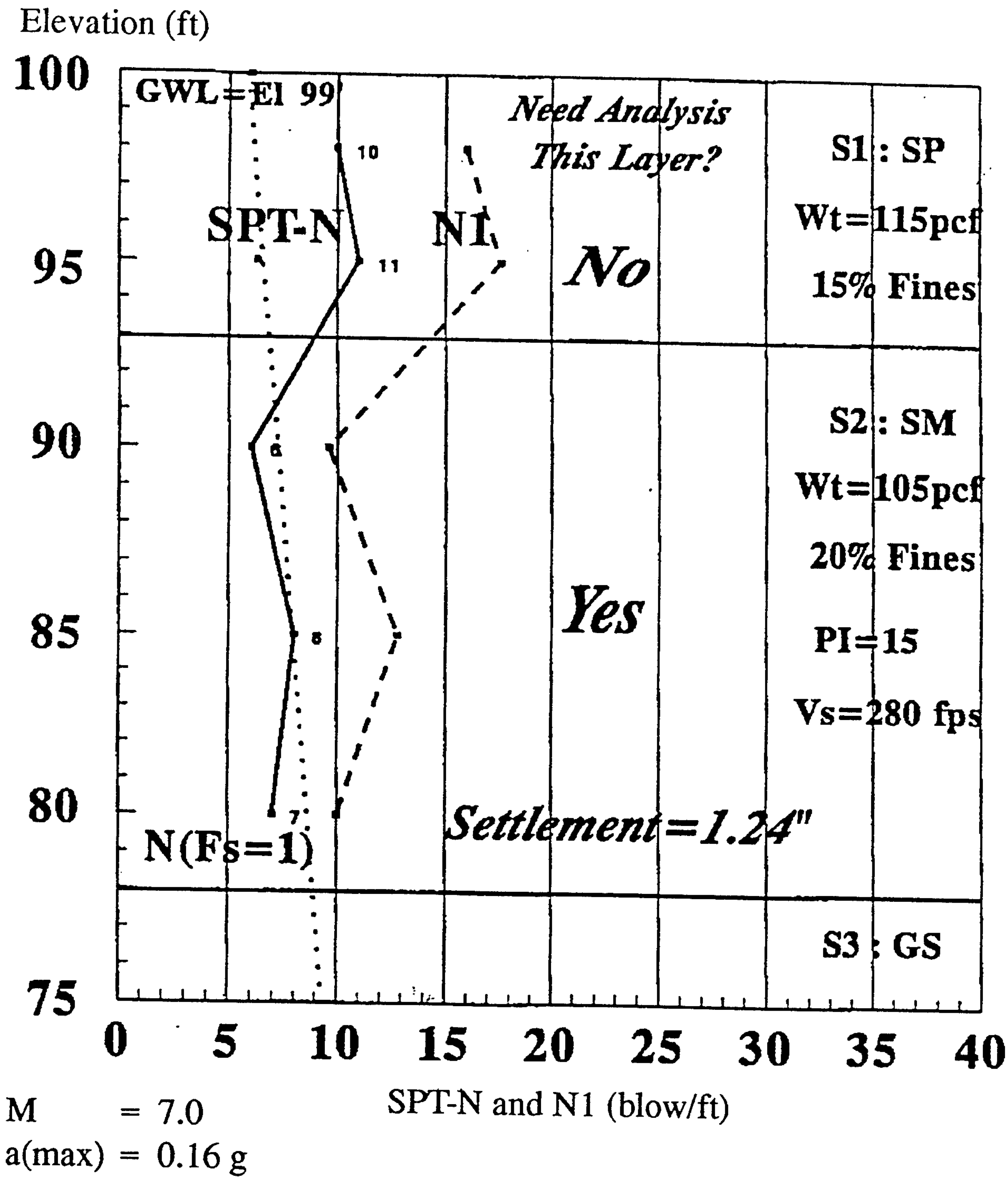

Figure 8. Results from LIQUFAC program.

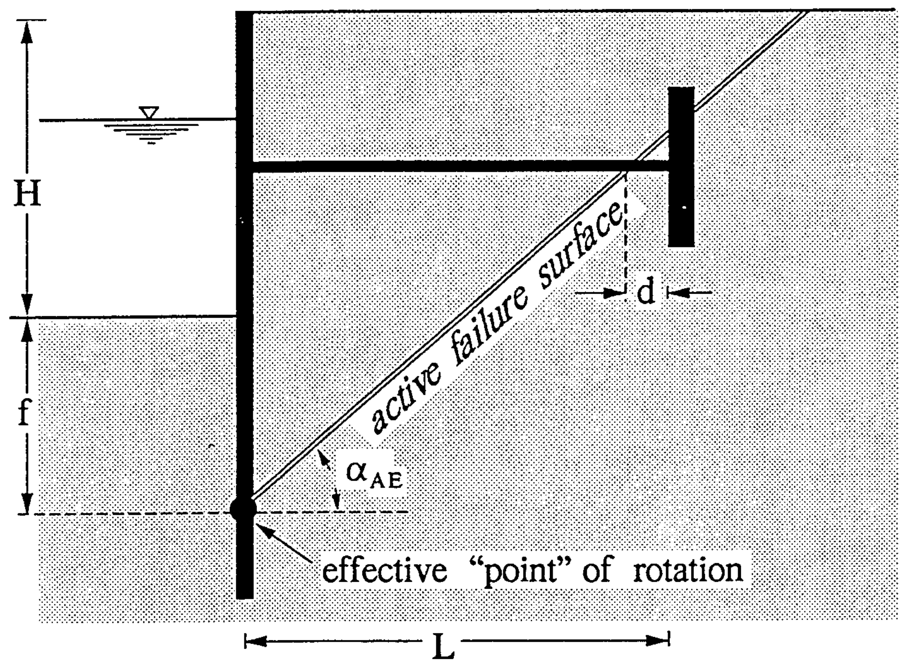

Figure 9. Sheet pile wall nomenclature. from Gazetas and Dakoulas, (1990) sponsored by National Science Foundation.

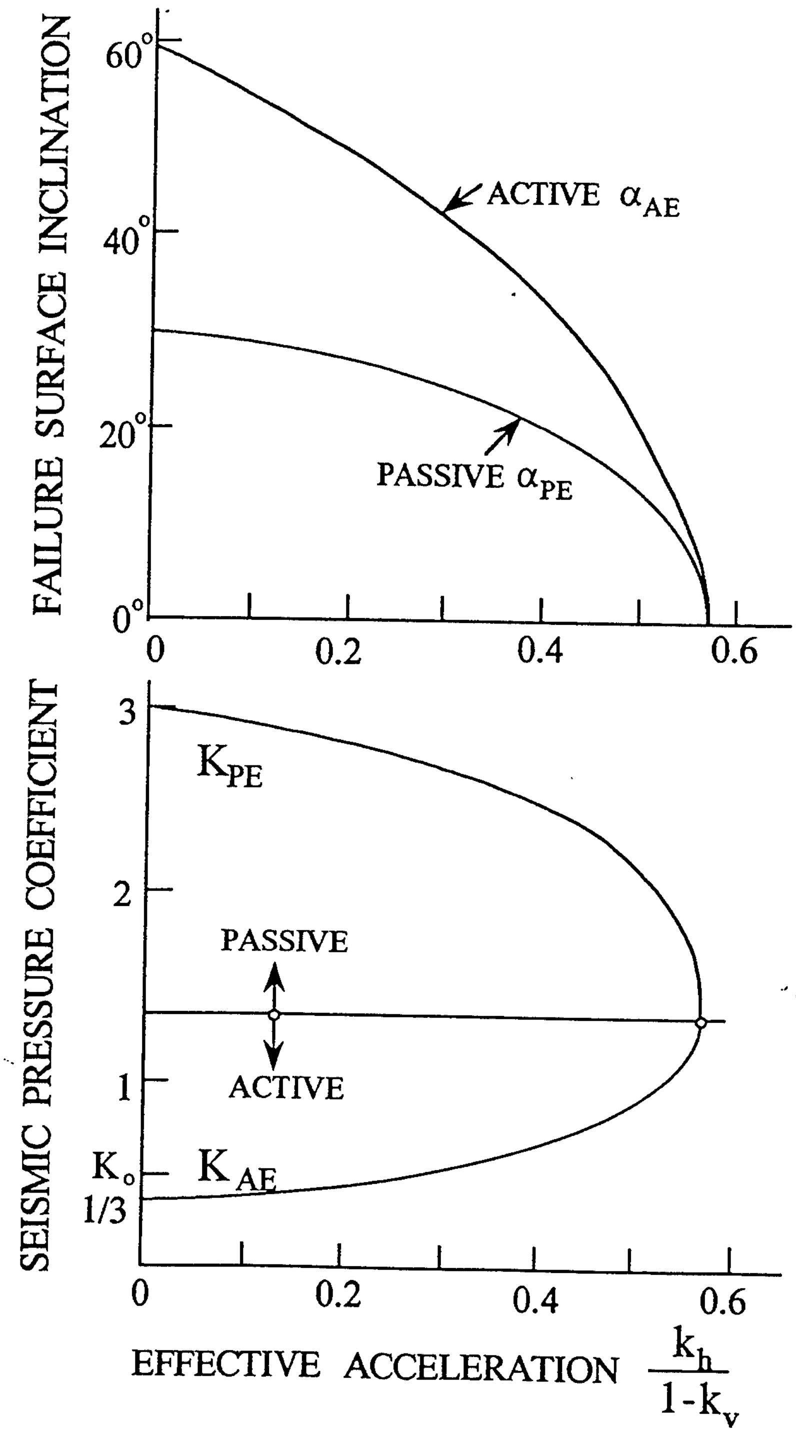

Figure 10. Failure surface inclination and pressure coefficients. from Gazetas and Dakoulas, (1990) sponsored by National Science Foundation.

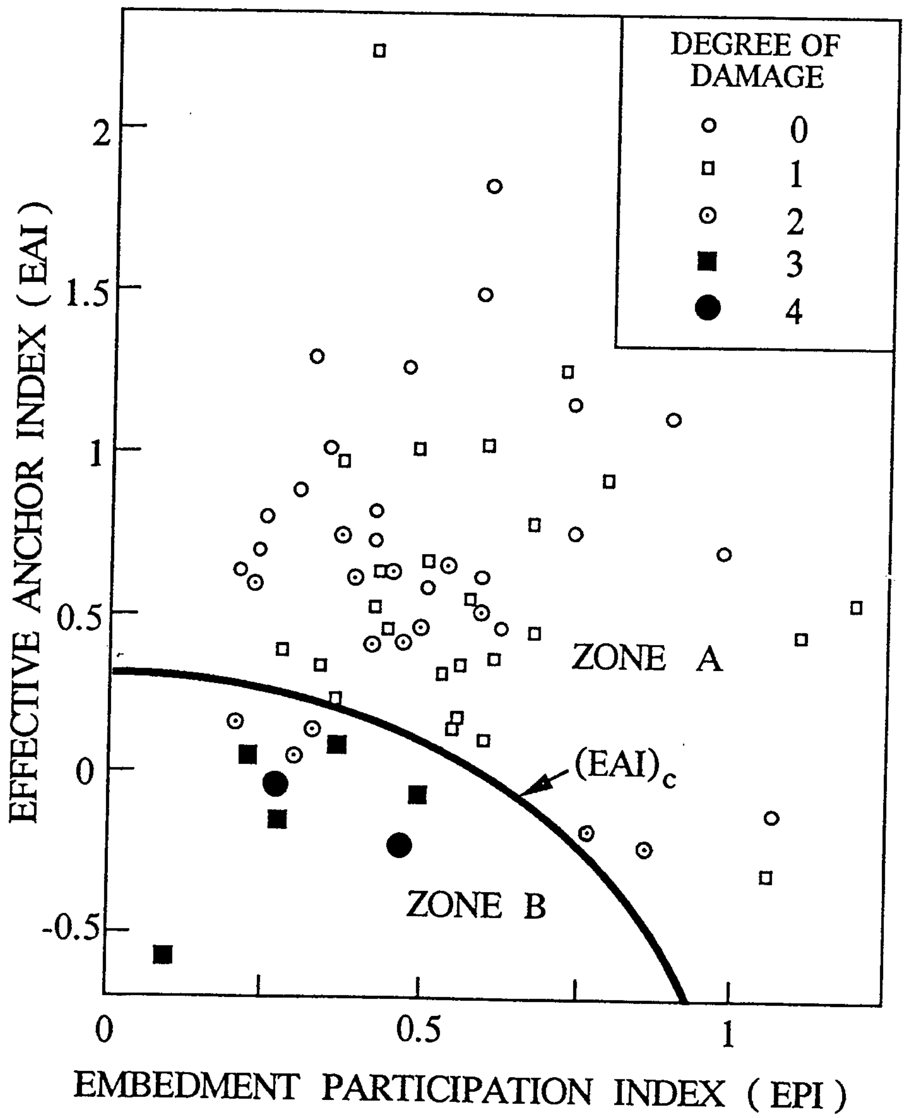

Figure 11. Sheet pile wall design chart. from Gazetas and Dakoulas, (1990) sponsored by National Science Foundation.

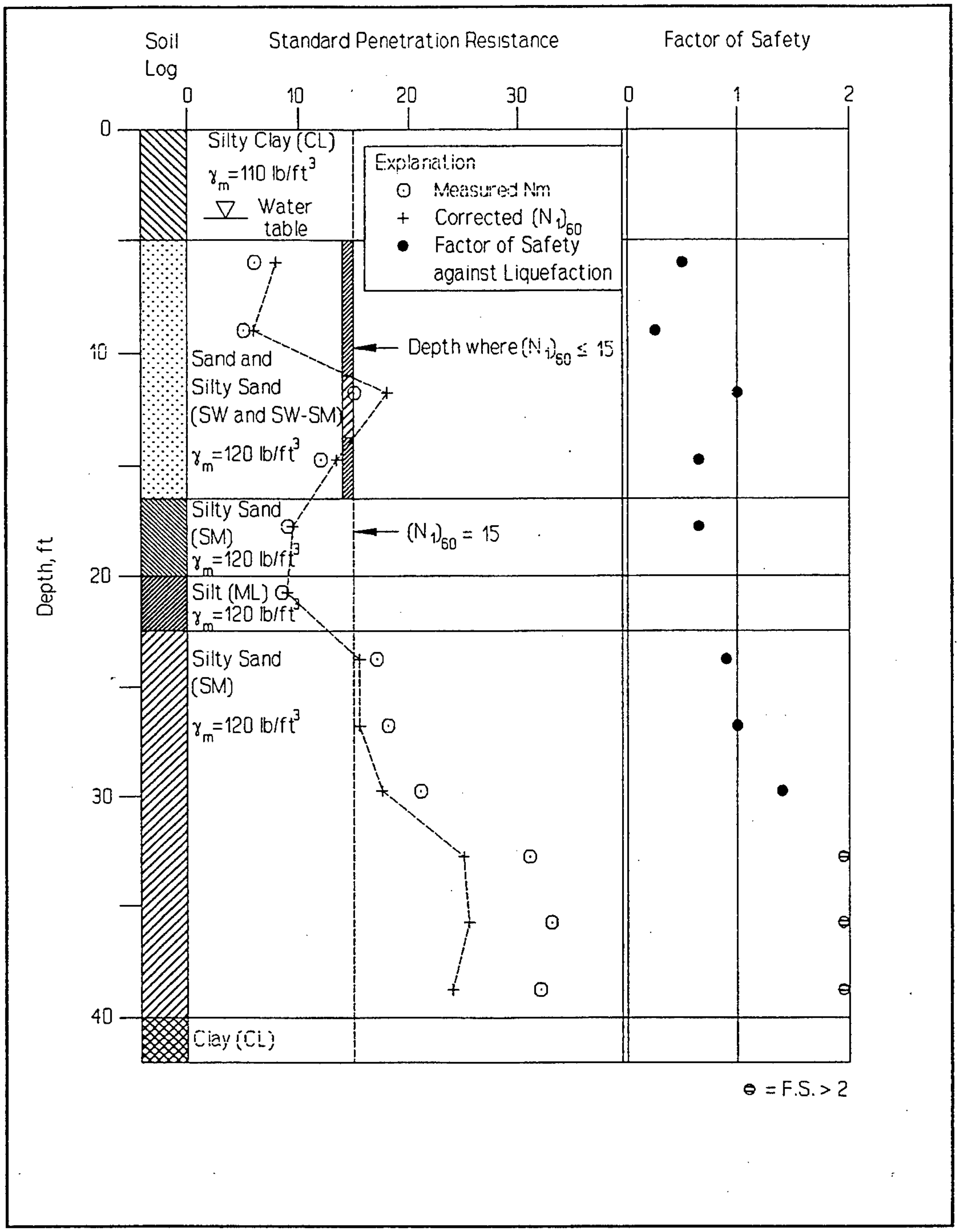

Figure 12. Boring log for example case.

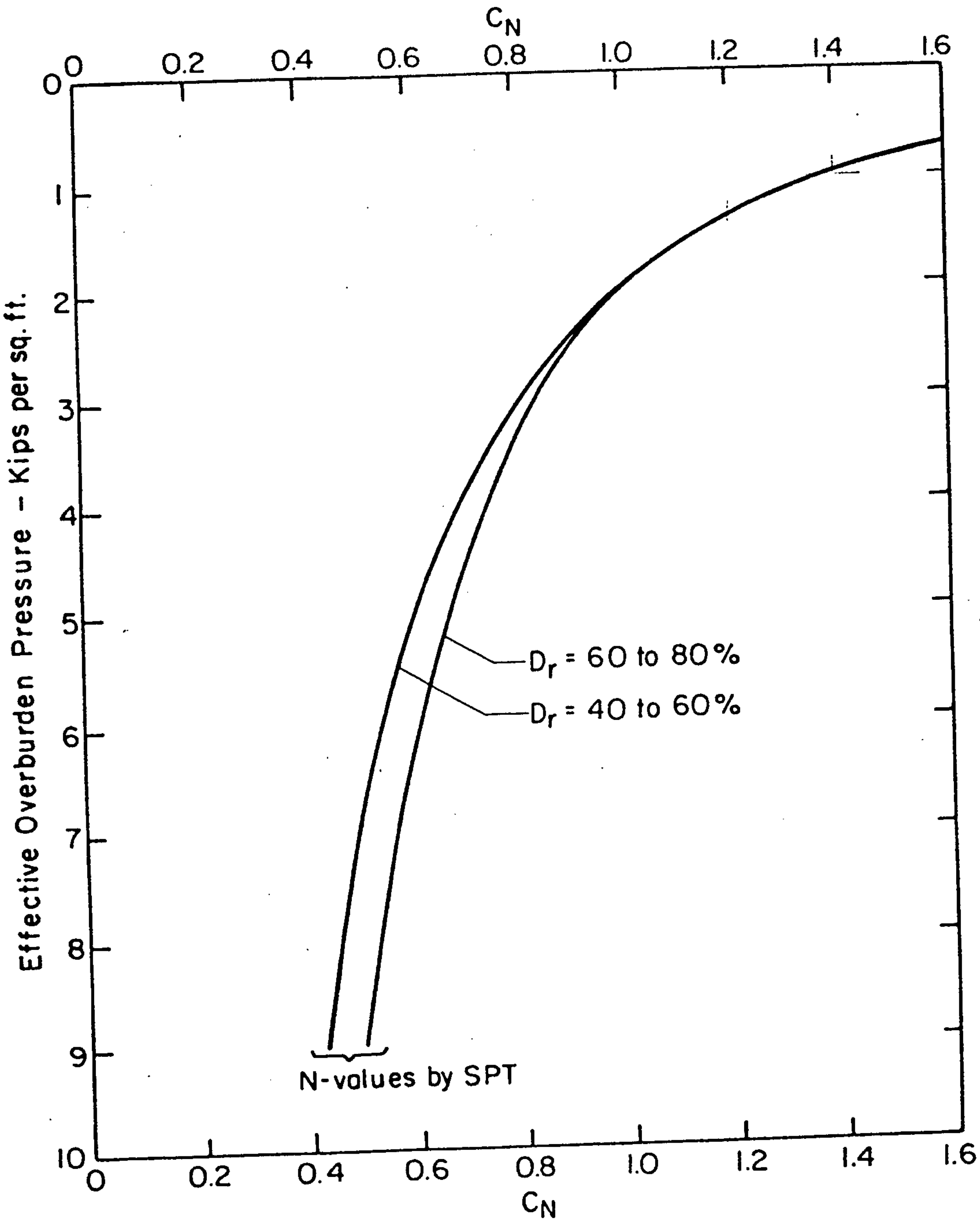

Figure 13. Effect of overburden on C_N.

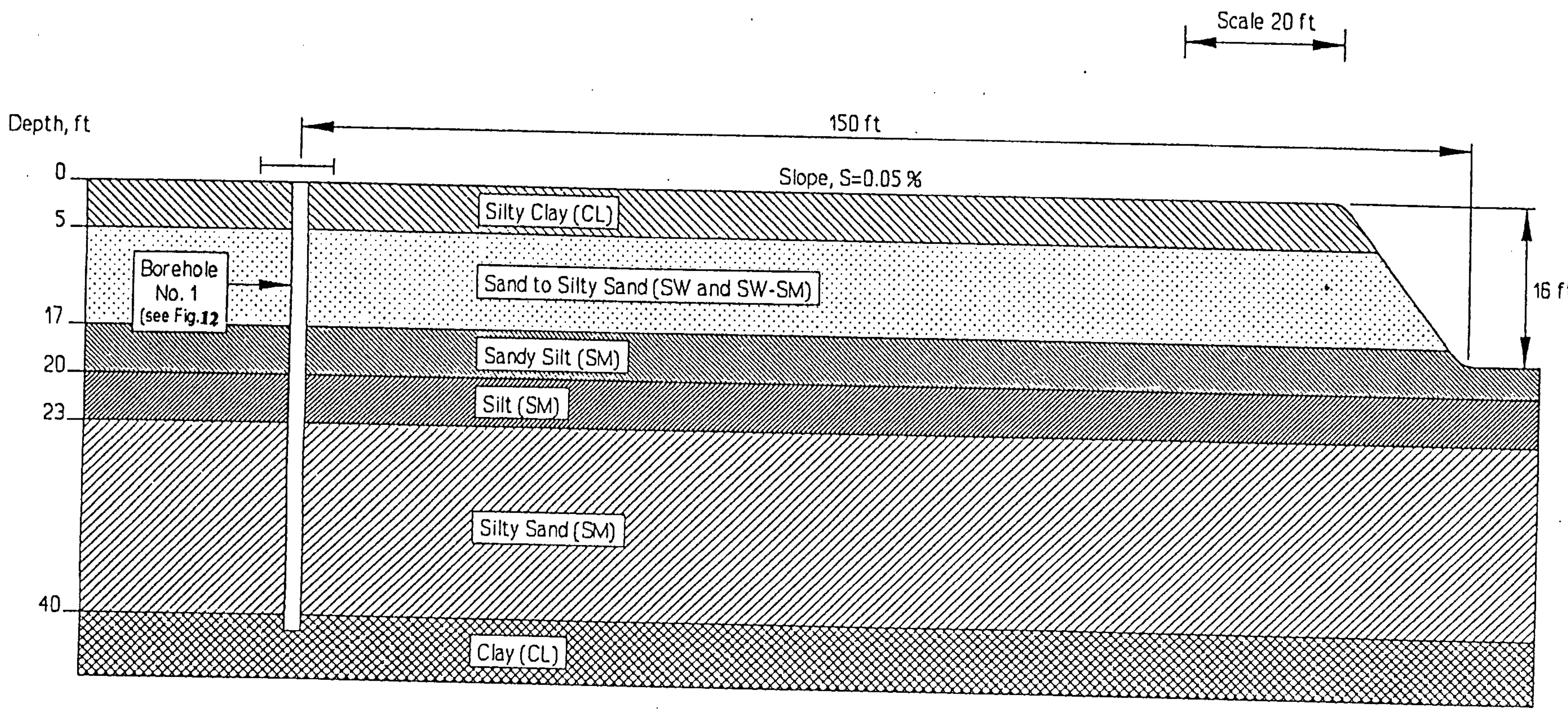

Figure 14. Site section profile for example.